Engineering Record

A Weekly Journal Devoted to Civil Engineering and Contracting

PUBLISHED BY McGRAW PUBLISHING COMPANY, INC.

Volume 70 NEW YORK, SATURDAY, JULY 25, 1914 Number 4

How to Name the Rocks

IT is not to be expected that the engineer should possess the specialized knowledge of the geologist, yet the very nature of his work in connection with tunneling and foundations of dams, bridges and buildings makes it imperative that he be able to recognize and classify the more common types of rock. It is not always an easy matter to do this offhand, and for this reason the condensed rules and simple field tests which Edwin C. Eckel describes on page 105 should be particularly valuable to every engineer, for sooner or later he will be called upon hurriedly to name the material upon which he proposes to found some structure. Of course, it is always possible to collect samples and have them analyzed by specialists, but during the preliminary stages in the location or execution of engineering works there may not be time for this and hence tests which will make known the nature of a rock quickly will find a wide sphere of usefulness. The determinations outlined by Mr. Eckel require no apparatus of any sort except a penknife, and a bottle of hydrochloric acid in some cases, but if the acid is not readily obtainable strong vinegar or lemon juice may be substituted. The condensed schedule of mineral characteristics is not formidable in length and should find a place in every engineer's notebook of reference data.

Concrete Surfaces Need Water

SPRINKLING concrete wearing surfaces on hot days must be practically continuous unless there is some moisture-retaining medium present. Recent observations of a number of pieces of road construction, on which contractors new to the work were engaged, indicate that the inspector is having his hands full in getting the "wetting-down" and "protection" specifications properly adhered to. Where earth is available a generous layer thrown on makes an excellent cover and holds water well. On the other hand, in two cases the "covering-with-earth" clause has been economically impossible of attainment because only gravel was available. A mere perfunctory scattering of a few shovelfuls of coarse sand and gravel is worse than nothing at all, because the evaporating surface is augmented and no protection is afforded. Some careful road builders in the West, recognizing the value of curing concrete under water, have made earth dams along the edges of concrete roads and divided the road longitudinally into a series of checks. In the floors of the Ford factory building described in this journal Feb. 14, page 182, wood shavings were spread. An ingenious chemical check on the sprinkling crew was discovered in this connection, for so long as the shavings were kept wet no stains appeared. Concrete is a chemical combination. Water is just as essential an ingredient as the cement and to insure proper setting clear to the top water must be present.

Fuel Oil in the Navy

THE forthcoming detachment of a number of battleships from the Atlantic fleet for service in the waters of the Pacific Coast is of interest from the fuel standpoint. The United States Navy is being steadily adapted to oil as a fuel, and it is estimated that at least 30,000,000 gal. will be required by the service during the next fiscal year. Sixty vessels in the navy, of which six are battleships, are now burning oil under their boilers. On the Pacific Coast this compact and clean fuel costs only 89 cents per barrel, against $1.39 on the Atlantic seaboard, and the difference is sufficient to explain the withdrawal of several ships from Eastern waters. All the new vessels are designed for oil burning, and the United States has been quick to appreciate the advantages of space, burden, and steaming radius accompanying the use of oil in British war vessels. Nearly 70 per cent of the world's fuel oil supply is produced in this country, which, fortunately, has fields in the East as well as the apparently inexhaustible areas of oil-bearing land in California. Strategically and economically the use of this kind of fuel has much to commend it and unless all signs fail the days of coal as the preferred naval fuel in North America are shortening.

Snails, Rabbits and Terminals

SNAILS and rabbits, according to George E. Hooker, civic secretary of the City Club of Chicago, are emblematic respectively of transportation in Chicago at the present time, with six steam-railroad terminals separated by gaps of various lengths, and the transportation possible with a comprehensive system of steam-railroad through routes. Mr. Hooker's antipathy for terminals in large stations is strikingly set forth in an ornate 89-page publication entitled "Through Routes for Chicago's Steam Railroads," issued by the City Club. He calls the hitching rail of the country town the original terminal and finds the mammoth passenger dead-end station as inappropriate for a large city with an extensive business district as the hitching rail would be. With diagram, photograph and red-ink epigram, interspersed with short paragraphs of text, he belabors the terminal, presenting arguments that have probably never occurred to the average reader. In the same graphic way he extols the through-route idea, with special application to Chicago. Possibly the expert terminal engineer could riddle some of the arguments and cause Mr. Hooker embarrassment in explaining how to overcome certain practical difficulties in his scheme. All in any way concerned in the subject, however, should find the book interesting and entirely free from the vagueness that characterizes many reports. Even a hasty perusal of Mr. Hooker's work will show unmistakably at what he is driving.

Croton Water and Fire Protection

IT will not be long before New York's water from the Croton Valley will be supplemented by the flow from the Catskill aqueduct, and the city is now considering plans for the most advantageous disposition of the various supplies. Croton water will be needed continuously for an indefinite future as a permanent asset. Its gravity pressure in the low-service district occupying the portion of Manhattan lying south of Seventy-second Street has long been insufficient. Pressures inadequate even for four-story buildings exist in some neighborhoods. Buildings very much higher are common throughout this portion of the city. House pumps and roof tanks are, therefore, the rule, with their entailed expense upon property owners or tenants. It is in the higher premiums for fire insurance, however, that the pecuniary disadvantages of the low pressure are most directly felt. Last winter the Merchants' Association of New York engaged James H. Fuertes to study the relation of water pressure to automatic sprinklers for fire protection. His report has just been made public, and is abstracted in the Current News section of this issue. The essence of the suggestion made in the report is to do for the whole district affected, at public cost, a service now privately performed at much greater expense for a part of this district. In other words, the proposal is to convert the Croton supply from a gravity to a pumped system. A substantial annual saving for the community is claimed. Pressures adequate for all buildings are not proposed, for manifestly in a district comprising many structures exceeding twenty stories a limit had to be set. Reasons for such a limit are many and obvious. Mr. Fuertes selected ten stories. Sufficient pressure for buildings of this height can, apparently, be provided at a reasonable cost; it is not excessive for existing street mains and plumbing; it meets the needs of a very large proportion of the buildings in the district; it is an effective pressure for fire fighting in all but the tallest buildings;

automatic sprinklers can be more generally used and will have a more reliable source of supply, and a material saving in insurance premiums will result. While in detail the problem is local, in principle it has a wide application. Fire losses are indirectly a general burden and all reasonable endeavors to minimize them should be encouraged.

Reducing Water Waste

A SUMMARY of statistics of water consumption in the metropolitan district of Boston in the Engineering Record of July 18 shows some of the results of the commendable efforts that are being made in many large cities of this country to reduce water waste. For several decades the water consumption in American cities has increased by leaps and bounds. For example, some recently published figures on water consumption in about 35 Massachusetts cities and towns show an increase from an average daily consumption of 46 gal. per capita in 1890, to 60 in 1900, and to 82 in 1910. In many cities, and especially in the large ones, the consumption has increased far beyond any of those figures and has assumed proportions that have made absolutely necessary the adoption of some measures which would result in a reduction of the waste. The reasons for this tendency toward rapid increase are many. Some of them are legitimate, others not, and it is gratifying to see this concrete example of actual betterment of conditions.

It is explained in the report mentioned that the large specific reductions were due partly to the increased use of meters and partly to unusually mild weather during the year to which the report applied. It is a well-known fact that the consumption, or rather the waste, of water in many cities is greatly influenced by the intensity of the winter, but aside from that the evidence in this case indicates clearly that there has been a progressive and certain reduction in waste, sufficient probably to postpone for a number of years large expenditures for construction that might otherwise have been necessitated.

Many other cities besides Boston have been following a similar policy in the last few years. About ten years ago there was a critical period of depletion in the supply from the Croton sources for New York. A vigorous campaign of waste reduction since that time has been one of the important factors that have contributed to make the Croton sources adequate for the needs of the city pending the completion of the works for additional supply from the Catskills. Washington, D. C., nearly a decade ago, undertook a campaign of waste reduction with results that were eminently successful. Chicago, Buffalo, Milwaukee and numerous other cities have, by means of additional meters or sub-surface investigations, secured promising results along the same line. On the other hand, although Toronto has spent much money and effort in this direction, the consumption has increased during the last few years at a rate almost unprecedented. Philadelphia, never favorably disposed toward meters, has accomplished some results by other means, one of which is also mentioned in a recent issue of this journal. This is the novel expedient of cutting down the night pressure in certain districts to a minimum assumed to be just above the point at which complaints would result. From the reported figures the result of this was striking and gives rise to arguments in favor of using as low a pressure in city mains as is consistent with good service. Other cities, notably Springfield, Mass., have had experiences that would tend to support the opposite argument, that is, in favor of high pressures to reduce the waste on the assumption that if the pressure is high enough any leak, no matter how small, will make itself conspicuous and will therefore be taken care of. The reports indicate that this is as true of underground leaks as of those on exposed piping systems.

Water waste, to the layman, is not usually one of the obvious shortcomings of the system. It is often obviously logical for the engineer to write into a report the necessity for and advantages of waste reduction and to make recommendations along that line. It is surprising, though, how many times such recommendations are passed over lightly, while at the same time large amounts of money are appropriated for the purpose of providing still more water to be wasted.

The saving of water such as is indicated by the Boston report is, therefore, a gratifying manifestation of true conservation. Efforts in this direction, vigorously carried out, can usually be relied upon to reduce waste in a greater or less degree, and this would seem to be one of the most promising lines of endeavor for waterworks managers at the present time.

Engineering Microscopy

WITHIN the last few years the microscope has attained a new usefulness as an instrument of engineering investigation. Its most notable applications in this field, or at least those of greatest interest to the structural engineer, are in the interpretation of physical performances of iron, steel and alloys along lines but little traversed by chemical analysis and particularly in the direction of ascertaining causes of metallurgical failures in materials of uniform or similar composition. Recent progress in physical chemistry, particularly a better understanding of the theory of solutions, has given the engineer a rational interpretation of the results of solidification of molten mixtures and a consequent explanation, more or less developed, of variations in the structure of alloys occasioned by changes in composition or treatment. The microscopic study of alloys is scarcely out of its infancy, and there is room for a vast amount of work in this field which will yield results of no little engineering value.

In the steel industry the microscope has become almost invaluable within its limitations of service, and while its applications are highly technical and most useful in the hands of the specialist, it is none the less desirable for the structural engineer to study this branch of metallurgical practice. Microscopy cannot usurp the functions of other testing methods, but it can supplement them most admirably, and in many instances has explained failures or unsatisfactory behavior of materials where chemical analysis disclosed little or nothing. The testing machine may show in a given case results in tensile strength and ductility which may in themselves be fairly good and yet below what might properly be expected in material of a stated composition. Under the microscope the steel may be seen to have been highly oxidized, with contamination by slag, the result being that the difficulty could not be corrected by ordinary heat treatment. Poor furnace practice produces results in the crystalline formation of material which can readily be interpreted by the experienced observer.

Composition is only one of the factors which determine the behavior of material and physical tests only show whether a certain steel is suitable for a general purpose or whether it is better than another steel, rarely enlightening the investigator as to the manufacturing modifications needed to produce the desired results. It is often impossible to deduce the factors responsible for failure from such tests, and an accurate idea of these frequently depends upon knowledge of the grain of the metal and the dimensions and forms assumed by the constituents. Thus, the remarkable difference between the properties of gray cast-iron and malleable cast-iron is due to the form and mode of distribution of the constituents, although both may have the same structural composition as expressed in percentages of ferrite and graphite. Practice in the use of the microscope throws open many hitherto baffling mysteries in the performance of structural materials and saves both time and material in the resulting discoveries.

Chemical analysis may indicate what the physical properties of a given steel should be when it is properly worked and heat treated, but one must fall back upon the microscope for the revelation of the structure which this sensitive material exhibits under different conditions of heating and cooling. Prof. Albert Sauveur pointed out last year before the American Iron and Steel Institute that a good metallographer cannot only give a reasonable estimate of the chemical composition, including oxygen, combined carbon in cast-iron and other points which analytical chemistry is only beginning to accomplish commercially, but he can go farther and predict the type of fracture with which the specimen will break. Metallographic examination has been most helpful in the study of malleable casting processes, in the control of the process of annealing, in the study of steel rails in relation to accident prevention, and in the selection of bearing metals combining a low coefficient of friction and sufficient plasticity to conform to the shape of the journal as the wear proceeds. The microscope is doing good service in assisting in the determination of zinc coatings for the protection of iron surfaces, in indicating the depth of penetration of the oxidation in cutting steel with the oxy-hydrogen blowpipe, and in the study of distortion in steel plates caused by punching. In the examination of complex ores, slags, ceramic materials, the study of insulators for extra-high voltage transmission, the detection of blast furnace slags sufficiently vitrified to serve as a raw material for Portland cement and in many other applications, engineering microscopy is coming into its own. By its use it is possible to make not only better products, but in cases where low cost is of controlling importance to make the cheaper grades of material within the specifications.

Extension of Payments on the Reclamation Service's Projects

INCLUDED in the so-called "development measures," upon which it is expected the President will concentrate his attention when anti-trust legislation is out of the way, is one extending the time of payment on the projects of the Reclamation Service. At present settlers must pay for their water in 10 years, or at an average rate of 10 per cent per annum. If the new measure becomes law—as seems likely in view of the support of the measure by the Administration, represented actively in this matter by Secretary Lane—the time of payment will be extended to 20 years. Neither under the present law nor in the measure pending is any provision made for payment of interest to the Government on the remainder of the debt.

It need hardly be said that either plan is very liberal, yet there have been criticisms—happily few—of the lack of liberality of the new measure. While it is correct to say that the settler pays the principal back to the Government but no interest, the bounty of the Nation can better be appreciated by saying that the settler under the proposed measure would pay 5 per cent interest and that he is made a present of the principal at the end of 20 years. With this statement before one the criticism of the average man will be not that the measure lacks liberality but that it is too liberal. It must be remembered, however, that the Government has always encouraged, practically by free gift, the development of the public domain into homesteads. It would seem, however, that the Nation has now gone a step farther than ever before in its liberality. Whether it has gone too far is largely a matter of opinion that will be influenced by one's locality and the degree to which one holds to the belief that the Government's dealings with individuals should be on a business basis. This journal is of the opinion that while interest might well be waived, especially since most of the funds have come from public-land sales, the 20-year repayment term is too long. Fifteen years would have been better, and the 5-year extension would have afforded the relief needed.

In connection with the relief necessary it should be noted that the extension measure is really being forced on the Government for its own protection and is not being adopted solely for the purpose of lightening the settler's annual load. Many have taken land on the reclamation projects with insufficient capital and experience. They have found it impossible to meet their payments, and unless there is relief they must abandon their lands. Not only will this be a serious matter for the man who has failed, but the Government must again go through the process of educating, aiding and encouraging an occupant of that tract. Relief has, therefore, been imperative. It is safe to say, however, that that man who cannot pay off his indebtedness to the Government in 15 years cannot do it at all. There will be exceptions, of course, but the statement is sufficiently general to stand.

While, therefore, the terms of the new measure are more liberal than desirable, some extension is necessary. There is such a thing as too great generosity, which saps initiative and makes people dependent. The measure before Congress errs rather in that direction than in lack of liberality.

Advanced Methods of Tunneling

THE two new subway tunnels from New York to Brooklyn together have a contract price of more than $13,000,000 and a combined length of about 9800 lin. ft. of double tracks. They are being driven partly through rock, but largely through sand, at a maximum depth of about 90 ft. below water level. Much of the material penetrated is difficult and treacherous to handle. The work is done under a narrow harbor channel subject at all times to a large amount of navigation, which makes obstruction by working plants prohibitive. The land terminals are in congested districts where space is valuable and little facility is afforded for storage and working room. These and other features make difficult the task of securing a safe and durable, permanent structure and of safeguarding the workmen during construction.

The precautions common to both tunnels, which have been made mandatory in the contracts, were described at some length in the Engineering Record of May 30, page 609, and include nearly all of the important provisions that have heretofore been made in work of similar character, together with some additional ones which were made imperative and subject to the control of the chief engineer instead of being, as is often the case, optional with the contractor. The important features included constant medical attendance, thorough medical inspection of compressed-air workers, general sanitary provisions, regulations of purity and temperature of air supply, fire protection, telephone equipment, protected passenger-elevator service, progressive stages of heavy air pressure, multiple air locks, and the provision of movable safety screens. Special provisions for the protection of property, most of them also having a bearing on the welfare of the workmen, include requirements for duplicate power supply, minimum compressed-air supply, and provisions for experimental work and recording important scientific data.

The Whitehall Street tunnel, described on page 92, is slightly longer and has a little higher estimated cost than the Old Slip tunnel, which is being built by the same contractors and under the same specifications and conditions. Most of its constructive features correspond with the other tunnel, except that at the Manhattan end the present and future connections for different land subways make an elaborate arrangement, involving intersecting alignments and complicated work between the pier and bulkhead lines, which will be executed with a single construction shaft in open water and a temporary transverse tunnel from which six different headings will be excavated. The excavation here is in rock and is of such great width that the heavy protecting concrete arch roof is subdivided into shorter spans supported on longitudinal walls. Beyond this point the tunneling is chiefly through sand, requiring a shield and pneumatic pressure. The segmental cast-iron lining has been designed of greater weight and strength than is customary, in order to eliminate or reduce the excessive distortion that has been occasioned in other places by exterior pressure.

At the Brooklyn end of the tunnel the shaft is of large dimensions and is constructed with double steel walls filled with concrete and designed to be sunk by the pneumatic-caisson process. A special feature of this shaft is the provision for shifting the roof of the working chamber after the shaft is sunk and thus giving a much greater vertical clearance to enable shields to be erected in the bottom of the shaft and driven through the sides for the construction of the tunnels in both directions, after which the roof can again be restored to the lower level until bulkheads are constructed in the finished tunnel sections. The common interests of the contractor, the owners and the public have been considered by the engineer in the provision of suitably prescribed sites for the location of the contractor's power plants and for the storage of his material on both sides of the river, and include the unusual item of an under-water storage for the large amount of clay that may be required in the event of a blowout and which might not be procurable from a distance during the winter months when navigation on the Hudson River is closed or impeded by ice.

The completeness and accuracy of the plans and specifications cannot fail to increase materially the economy and rapidity of the work and the safety of the workmen. Their practical value is illustrated by the completely detailed and dimensioned contract drawings for the Brooklyn shaft, which were intentionally made adequate for working drawings. The specifications are a fine example of an able digest of precedents combined with competent modifications and extensions that eliminate many frequent sources of danger, uncertainty, delay and expense, and materially benefit all interests concerned.

Whitehall Street Tunnel, New York City

Twin Concrete-Lined Cast-Iron Tubes 6500 Feet Long, with 300 Feet of Double-Track Concrete Arch at Manhattan End

IMPORTANT features of one of the dual-system subway tunnels under the East River in New York City, the Whitehall Street tunnel, are the complicated arrangement of New York approaches and shaft, difficult excavation, including a rock tunnel 77 ft. wide with long-span concrete-arch roof, a reinforced-concrete shaft in open water, pneumatic-caisson shaft sinking, variable-section cast-iron lining rings, cast-iron sump chamber, special provision for the installation of contractor's plant, service required and compressed-air specifications.

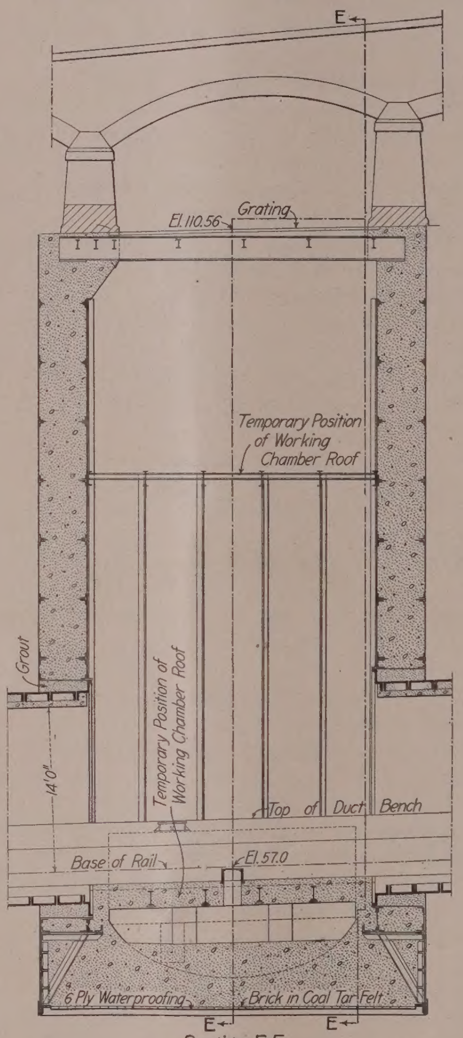
This tunnel from Whitehall Street, Borough of Manhattan, under the East River to Montague Street, Borough of Brooklyn, New York City, is contract section 2 of route 33 of the Rapid Transit Railroad, and comprises a total length of about 6800 lin. ft. of double-track tunnel excavation to be conducted in compressed air except where the work is above water level or in solid rock. Except for 300 ft. of double-track concrete-arch tunnel in the rock excavated at the Manhattan end the structure will consist of two cast-iron single-track tubes lined with concrete and excavated under pneumatic pressure. There is to be a permanent ventilating shaft in New York sunk by the open-cofferdam or pneumatic-caisson method and one in Brooklyn sunk by the pneumatic-caisson method, both being used for construction purposes. It is expected that the U. S. Government will remove Coenties reef, which overlies the tunnel at the Manhattan end, previous to the construction of the tunnel, so as to obviate any danger to the completed structure due to future blasting when the channel is deepened.

Geological Conditions

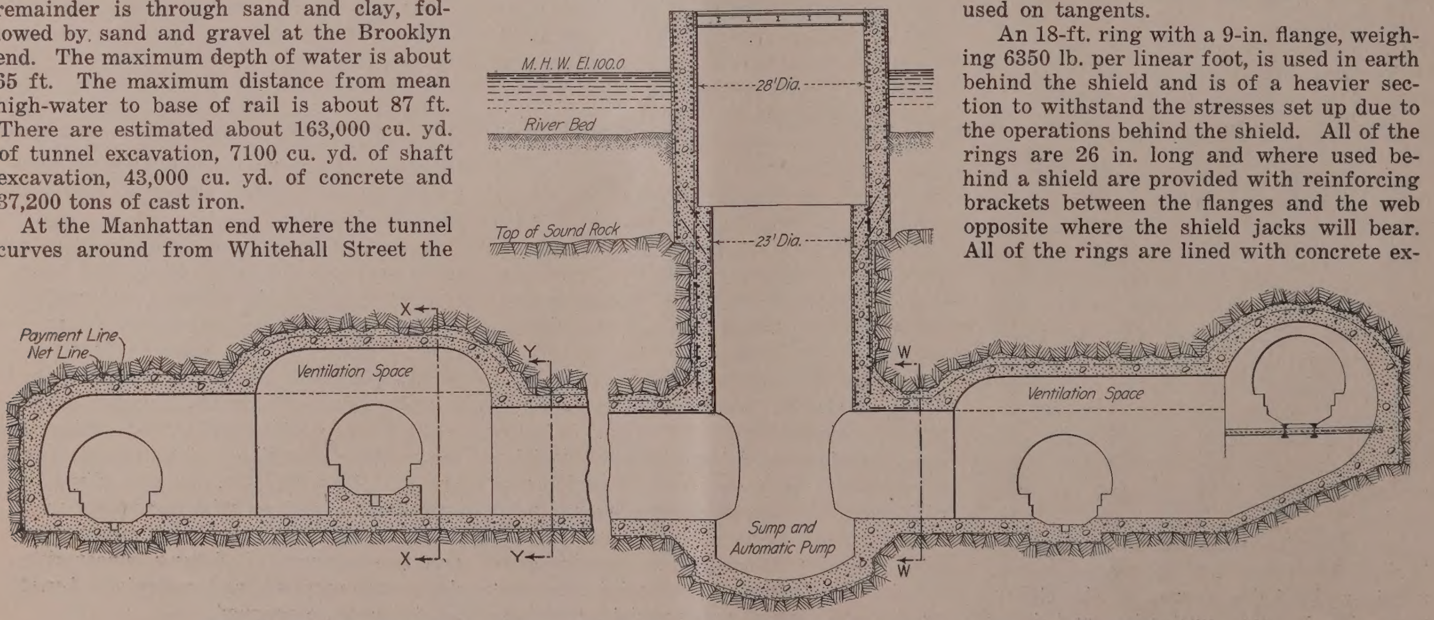
The structure is 2560 ft. long between pierhead lines and 4089 ft. long between centers of New York and Brooklyn shafts. Beginning at the Manhattan end the excavation commences in Manhattan schist rock, then extends about 500 ft. through sand and clay, then penetrates 400 ft. through a pinnacle of Fordham gneiss, after which the remainder is through sand and clay, followed by sand and gravel at the Brooklyn end. The maximum depth of water is about 65 ft. The maximum distance from mean high-water to base of rail is about 87 ft. There are estimated about 163,000 cu. yd. of tunnel excavation, 7100 cu. yd. of shaft excavation, 43,000 cu. yd. of concrete and 37,200 tons of cast iron.

At the Manhattan end where the tunnel curves around from Whitehall Street the construction is complicated by provisions for future connections to the Broad Street subway and tunnel to Atlantic Avenue, Brooklyn. This requires increased arch spans at the junction points in each case and necessitates carrying track 3 under track 2 to avoid a grade crossing, as indicated in the general plan. It is expected that all of the tunnel excavation at this point will be in rock and it is contemplated to insure watertightness by a cast-iron lining except for a distance of about 170 ft. at the widest part, where the four tracks converge and for a distance of about 80 ft. at a point near the foot of Whitehall Street, where concrete construction is adopted for the above-mentioned connections.

At this point the rock roof will be supported on concrete arches, without reinforcement, carried by reinforced-concrete longitudinal walls, as indicated in the section diagram. No waterproofing will be laid over the extrados of the arches, but watertightness will be secured by using a dense concrete and by grouting. The upper part, at least, of the Manhattan shaft located in open water adjacent to pier 4 will be built of reinforced concrete designed to resist possible impact from boats in the slip. Drifts will be excavated both ways from the foot of this shaft to provide for tunneling two pairs of tubes in both directions, making eight headings operated from it, as indicated in the general vertical section through the shaft.



STEEL-LINED SHAFT, BROOKLYN

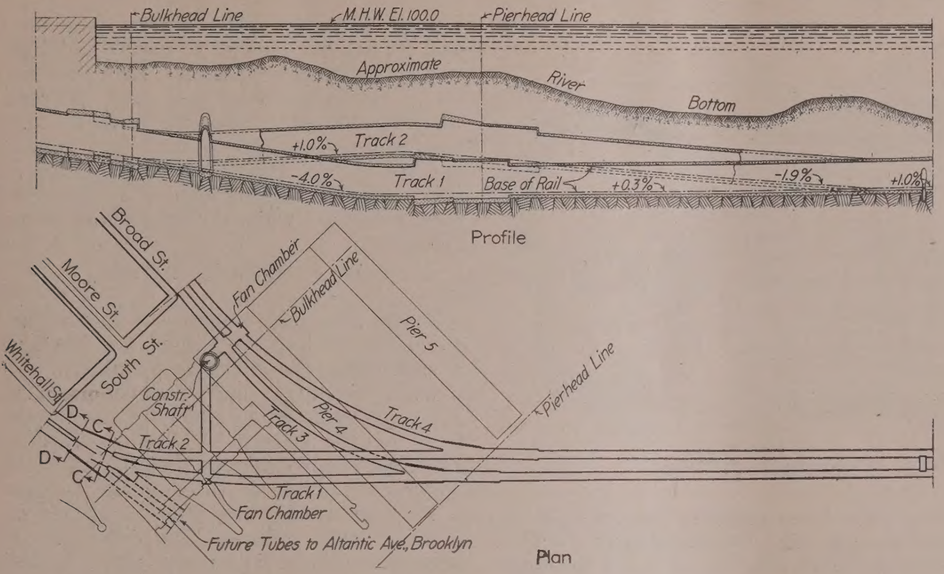


TEMPORARY CONSTRUCTION SHAFT AND TRANSVERSE CONNECTING TUNNEL AT MANHATTAN END

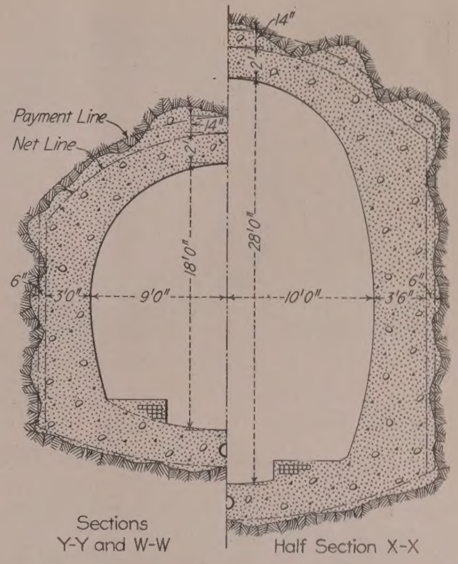
Cast-Iron Lining

The cast-iron lining is of the sectional flanged-ring type ordinarily used for shield-driven tunnels. In the rock, where it is not expected to use a shield, there are three different sizes of a lighter section than that to be used in earth behind the shield. The 23-ft. ring with a 9-in. flange, weighing 5890 lb. per linear foot, is introduced at the junction on the transition from the regular ring into the concrete-arch construction. The 18-ft. ring with a 7-in. flange, weighing 4060 lb. per linear foot, is used on curves in the rock to provide greater clearance, while the 17-ft. 2-in. ring with a 7-in. flange, weighing 3800 lb. per linear foot, is used on tangents.

An 18-ft. ring with a 9-in. flange, weighing 6350 lb. per linear foot, is used in earth behind the shield and is of a heavier section to withstand the stresses set up due to the operations behind the shield. All of the rings are 26 in. long and where used behind a shield are provided with reinforcing brackets between the flanges and the web opposite where the shield jacks will bear. All of the rings are lined with concrete ex-



PLAN AND PROFILE OF VARIOUS TUNNELS AT MANHATTAN END



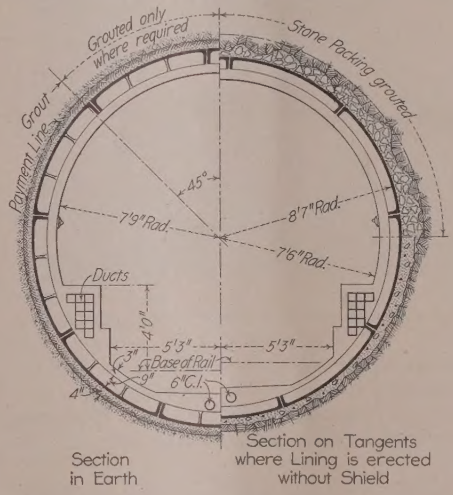
TYPICAL CROSS-SECTIONS IN ROCK

tending 6 in. inside of the flanges, thus making a total thickness of concrete of from 12 to 14 in. inside the cast iron.

The earth section has a clear inside diameter of 15½ ft., and the top for a distance of 45 deg. each side of the center line is not grouted unless required by the engineer. In rock the clear diameter is 15 ft. 10 in. on curves and 15 ft. on tangents. In rock where no shield is used the cast-iron tube is supported below the springing line on a concrete cradle and above the springing line it is backfilled with stone packing, grouted. Where a shield is used in rock the invert of the tube rests on a concrete cradle 10 ft. wide and the remainder of the circumference of the tube is grouted and backfilled with stone packing where practicable.

Each ring is made of nine nearly equal segments and a narrow key. Each segment is provided with a 1½-in. grout hole and screw plug and the flanges are secured by 1⅛ or 1⅜-in. bolts with rolled threads. The head and nut washers bear on grommets of hemp and red-lead paste engaging the countersunk bolt holes. The 23-ft. arch rings are 30 in. long and the details are similar, but the metal is lighter. In all cases allowance is made for clearance and excavation in rock is paid for outside the neat line of the cast iron for a distance of 12 in. above and 6 in. below the springing line. In earth an allowance of 4 in. outside the neat line is made.

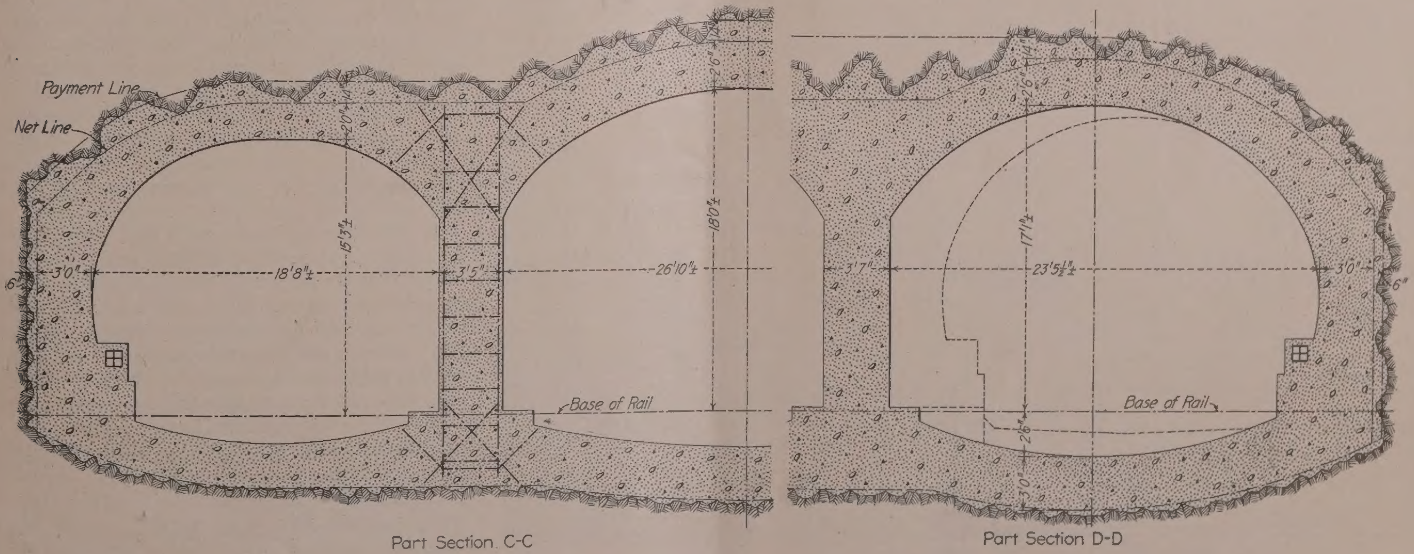
In one of the low points of the tunnel, near the center of the river, the tunnel tubes, 42½ ft. apart on centers, are connected by a transverse passage with a cast-iron lining of 1-in. thickness and 11-ft. diameter, similar to the tunnel lining. The upper part of this lining is regular and cylindrical and the lower part is extended to a depth of 12 ft. below the tunnel center with vertical sides and concave invert and is lined with concrete to form a sump 6 ft. wide, 5½ ft. high and about 20 ft. long. This will receive the drainage from both tunnels and discharge it through pumps located in the passage above. Although it is expected that this sump will be constructed in rock excavation, care was taken to make the cast-iron lining of a simple and regular pattern that enabled it to be easily assembled and is also suitable for use in earth. At another point near the middle of the river the two tunnel tubes are connected by a transverse feeder chamber 11 ft. in diameter with a construction similar to that of the sump chamber, except that it is symmetrical around the horizontal center line, the halves being like the upper part of the sump chamber.



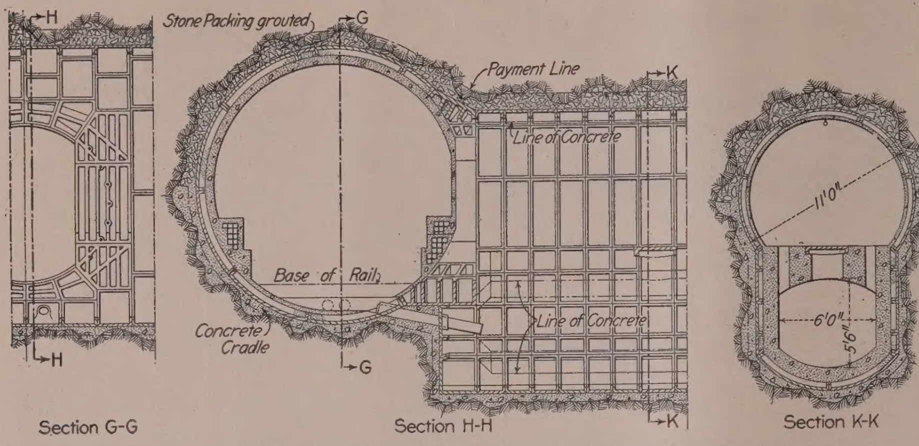
SECTIONS LINED WITH CAST IRON

BROOKLYN SHAFT

The shaft at Furman and Montague Streets in Brooklyn is sunk through the masonry-arch viaduct of Montague Street, and its walls will later support one span of that structure. The shaft has a double steel lining filled with concrete and sunk by the pneumatic-caisson method, being similar in design to the shafts used for the Pennsylvania Railroad tunnels in New York. The



HALF-SECTIONS OF THREE-ARCH AND TWO-ARCH CONCRETE STRUCTURE FOR WIDE TUNNEL IN ROCK

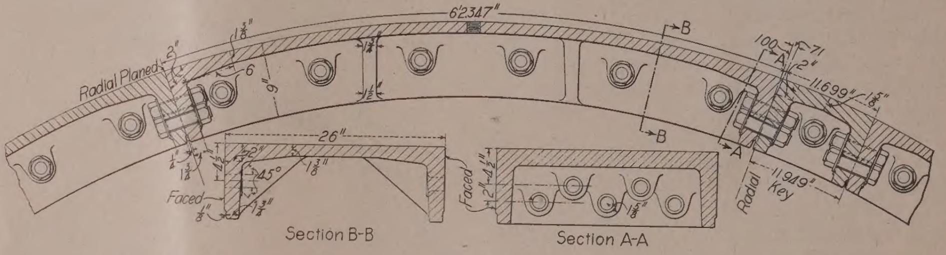


DETAILS OF CAST-IRON LINED SUMP CONNECTING TUNNELS

shaft is divided by a center partition into separate working chambers for the two tunnels. The floors of the chambers are waterproofed and covered with a deep layer of concrete, in which are formed inverts for the bottoms of sumps. These sumps are drained by automatic pumps and covered by the tunnel floor, which is supported on I-beams.

The exterior skin plates of the shaft lining are calked from top to bottom and the interior ones are calked in the lower portion of the shafts. In both inner and outer skins bulkhead frames and plates are provided to close temporarily the openings in the wall which afterward provide openings for the egress of the tunnel shields. Completely dimensioned details of all the steel work in the shaft lining are given in the specifications with the intention of thus providing working drawings which will serve the contractor for the immediate construction of the shaft and enable this initial work to be undertaken without loss of time after the award of the contract.

The detailed drawings show two positions for the roof of the working chamber, which is composed of convex plates riveted to the flanges of transverse horizontal plate girders. The lower position, about 8 ft. above the cutting edge, is intended to be used while the shaft is being sunk, and the upper position, about 25 ft. higher, shows the location to which the roof will be moved after the shaft is sunk, thus permitting pressure to be maintained in the lower part while the bulkhead closing the tunnel entrances is removed and until the tunnels are constructed far enough for the installation of airlocks. After this the roof will be removed and materials lowered into the shaft for the construction of shields to extend the tunnels in the opposite direction. Then the roof will be again replaced, the shields erected, and construction of the land tunnels carried far enough to permit building air-lock bulkheads in them. After the exits of the land shields are built the roof will be retained in place until one air-lock bulkhead is installed in each land tunnel and two in each river tunnel. Provision is also made for replacing any of the chamber roofs necessary while the tunnels are being driven.



DETAILS OF FLANGED SEGMENTS OF TUNNEL LINING

WATERFRONT FACILITIES

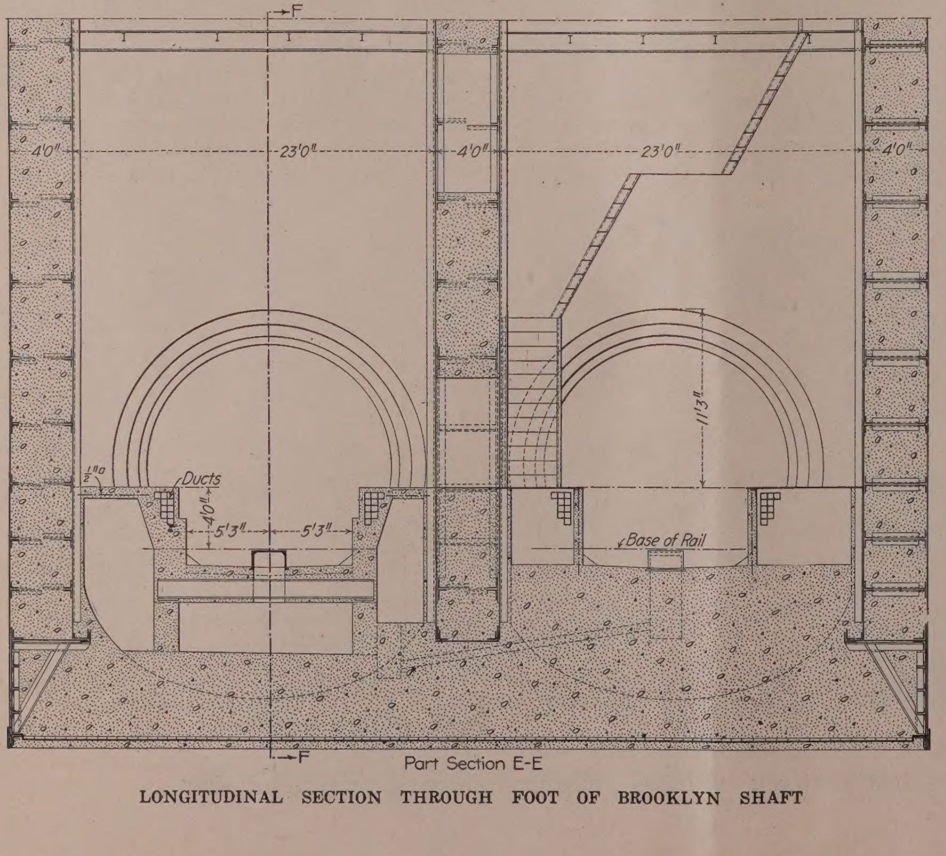
Spaces have been provided in Manhattan and Brooklyn for the receipt and storage of materials and for the installation of steam-generating and air-compressing plants. These are accessible to the waterfront for the delivery of coal and involve pipe lines about 1300 and 2500 ft. long. An important feature of the material storage at Montague Street and Pier 15, Brooklyn, is the facility afforded for depositing a large quantity of clay under water in the slip there so as to provide abundant storage for use in the winter when the boats may be icebound and the clay can be dredged up and delivered to the tunnels as required, without delay and at a cost not greatly larger than required for unloading from barges. It is probable that a portion of this slip may be decked over and provide for service tracks and coal handling.

CONSTRUCTION METHODS AND WATERPROOFING

The tunnels and shafts will be lighted by electric lamps of the equivalent of 32 cp at 32-ft. intervals. The pumps for operating the tunnel-shield jacks must not be located in the heading chambers. Voids outside the cast-iron tunnel lining will be filled with grout discharged under an air pressure of 60 lb. per square inch in excess of the external water pressure. Grout for the dry stone packing forced through the lower holes of the lining must show at the upper holes before the latter are connected for grouting. Where the tunnels in earth are adjacent to buildings all voids outside the lining must be grouted to the top of the tunnel, but in other cases, except in rock, the upper part of the tunnel need not be grouted.

Where the shield is used the grout will be made with natural cement and sand in order that it may not set too quickly and impede the movement of the shield. In other cases equal parts by volume of Portland cement and concrete sand fine enough to pass through a sieve with sixty-four openings per square inch will be used.

All joints between the flanges of the tunnel-lining segments will be cleaned with high-pressure air or water jets or by scraping, calked with metallic lead, and afterward painted with 1:1 cement mortar before the lining is concreted. The floor of the caisson for the Furman Street shaft is to be waterproofed with six piles of



LONGITUDINAL SECTION THROUGH FOOT OF BROOKLYN SHAFT woven fabric saturated with pitch and combined in triple layers before delivery to the caisson.

SAFETY PROVISIONS

Cages with strong protected roofs are to be installed in the shafts for hoisting men and materials and must be provided with safety catches, bronze bearings, landing dogs at all landings, and devices for the prevention of overwinding. These safety devices must be tested at installation, and every three months thereafter. Signal devices must be provided to give instant communication from the foot of the shaft to the hoist room, and the cage openings are to be protected by automatic gates or trap-door covers.

Unusually complete regulations are specified for the compressed-air work and plants. These specifications were fully abstracted in the Engineering Record of May 30, page 609. These provide for exceptionally large power plants and air delivery, for fireproof construction, sanitary quarters, surgical and hospital attendance and facilities, purity of the compressed-air supply, the ventilation of the tunnel, and for safety appliances in the tunnel. Where the heading pressure exceeds 22 lb. per square inch, bulkheads and air locks are required to separate it into equal stages, and an emergency lock is provided in the upper part of the bulkhead which shall be always left open toward the heading; one of the main locks is arranged to be locked in from the outside. Movable steel safety screens extending from the springing line to the top of the tunnel are to be maintained not more than 115 ft. in the rear of the shield, and are to be accessible by an elevated runway from the shield platform.

This work has been designed by and will be executed under the direction of the Public Service Commission, of which Alfred Craven is chief engineer and Frederick C. Noble division engineer. The joint bid of Booth & Flinn, Ltd., and the O'Rourke Engineering Construction Company was the lowest received when the bids were opened May 22.

Concrete Mixer for Cleaning Contact-Bed Stone

ALL of the crushed stone in the primary contact beds of the sewage-treatment plant at Plainfield, N. J., was removed last year, washed in a concrete mixer, and passed through a revolving cylindrical screen with ¾-in. holes. The operation is described in the 1913 annual report of Francis E. Daniels, director of the water and sewerage inspection division of the New Jersey State Board of Health. A stream of water was applied through a perforated pipe at the center of the revolving screen. The dirty stone was dumped into a hopper and dropped into the mixer. It passed through the mixer and revolving screen and was discharged into a cart below. Such stone as passed through the holes of the screen was rejected as being too fine, and enough coke was placed on the beds to make up for the volume thus lost.

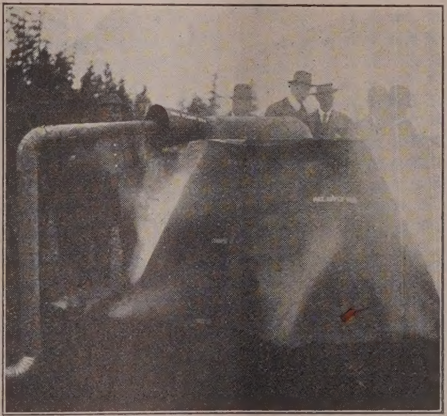
TOKIO'S MUNICIPAL WATERWORKS, which are reported to bring in an annual net profit of nearly $500,000, have lately received government sanction for new work, to cost $10,360,000, which will provide a system capable of supplying a population of 3,000,000.

Stump Burning to Reclaim "Logged-Off" Lands

Methods and Costs of Clearing Tracts for Agriculture in Pacific Northwest

By LE ROY W. ALLISON
Newark, N. J.

AN engineering problem of comparatively recent origin is that of adequate land clearing, particularly as found in the Pacific Northwest. In this vicinity what is known as logged-off lands are covered with fir and pine stumps of cut-down forest trees, varying from 3 to 6 ft. in diameter. These are intermingled with a succeeding growth of similar trees up to about 1 ft. in diameter, and are covered with an undergrowth of willow, vine maple, hazel, laurel and the like.


STUMP BURNER IN OPERATION

The question of effective reclamation of such property is one that may be consistently compared to the problems of swamp drainage and desert irrigation in other parts of the country. With proper clearing these logged-off lands are found to be very valuable for agricultural enterprises, being extremely fertile owing to the many years' accumulation of a thick deposit of forest mold. It is essential that this mold, forming a rich top soil, be preserved intact, without an additional infertile subsoil deposit over the surface, leaving the ground in a suitable condition for immediate cultivation. Each section of land presents its individual problems relating to the economical and adequate elimination of the incumbrances.

METHODS AND COST OF CLEARING

Many methods have been employed for freeing the lands from stumps and preparing for seeding and cultivation, such as grubbing, burning, charpitting, blasting and pulling either by horse machine or donkey engine. Of these the burning out of the roots by charpitting is the most favored method, for it can be applied by almost anyone of average intelligence. It has a great disadvantage, however, in being a slow and tedious operation.

The blasting of stumps entirely by powder and the shattering of the subsoil as a means of producing numerous cracks for water storage and restoring the fertility are frequently advocated and have been found effective. But for specific land clearing the resultant scattering of unproductive subsoil over the surface is not altogether satisfactory for uniform fertility of area, while only those accustomed to handling explosives can use the method with entire safety. After the charge is fired the disposal of the freed stump sometimes becomes an important item, as is evidenced in the piles of partially burned stumps and roots surrounding cleared areas. Not only are they unsightly, but they are dangerous to stock and tend to promote the growth of weeds. Many infertile spots remain when placed under cultivation, as well as large holes to be filled with fertile and valuable top soil.

The cost per acre of clearing lands varies considerably with the character of the subsoil, condition of land, etc., but a fair average basis for this part of the country may be taken as follows:

Method	Average cost per acre
Powder and horse puller	$110
Powder and team	100
Donkey engine	90
Powder and grubbing	80
Powder and burning	70

A complete land-clearing plant has recently been devised and placed in service which overcomes some of the disadvantages of the various methods noted and offers an interesting solution of the problem. It combines the different generally used systems into one plant and affords through five individual features twelve particular combinations for operation, thus meeting almost any condition of land and expenditure.

This plant is based upon the unit principle and is known as the Blake land-clearing machine. Operated by a gasoline engine, it consists of a five-fire Pluto stump burner, a cordwood saw, a power grubber to use with stump pullers, a geared horse stump puller and a power stump puller.

STUMP BURNER

One of the most interesting features of the plant is the stump burner, which is operated by a blower on the machine. A line of pipe connects the blower to a sheet-iron hood by means of a gooseneck, entering the hood through an aperture in the top. This hood is made in four sections and is set over the stump to be demolished.

The use of the burner is particularly simple. A hole is bored down through the center of the stump and a small stick of powder is employed to split it apart. This splitting, while not entirely essential, facilitates in allowing the fire started on top of the stump to gain rapid headway and burn more evenly. The hood is then placed over the stump and banked up with a little earth at the bottom and forms a closed but not airtight chamber. The fire is then started and pipe connection from the blower made, after which a constant downblast is blown upon the flame, continually fanning it.

Stumps are consumed in this manner in from 2 to 4 hr., depending on the size and nature, while those of excessive diameter and unfavorable condition may require a longer period. During the operation of the machine the heat within the chamber becomes intense and the sheet iron red hot, making a veritable charpit. The hot air rising from the hood creates a partial vacuum, drawing in the cold air around the outside of the hood, thus preventing it from burning out.

Upon removal of the hood following the consumption of the stump the fire is covered with dirt, allowing the roots to charpit to the ends without the aid of the blower.

Official tests of the burner apparatus have been made, showing effective results at low cost. At Gladstone, Ore., an old and water-soaked stump 20 ft. in circumference at the base, 13 ft. in circumference at the top and 4 ft. high was used for test purposes.

The equipment used consisted of a 4-hp gasoline engine, a No. 3 Buffalo pressure blower and a line of 4-in. pipe, with hood and gooseneck. A 3-ft. vertical hole was bored in the stump from the top with a 1-in. auger to accommodate a stick of dynamite used for splitting. Confining the heat by the sheet-iron hood caused rapid combustion of this heavy, water-soaked timber. The stump was consumed in 6 hr. The total cost was $1.15, divided into 75 cents for gasoline for engine operation and 40 cents for explosives.

The method of operating a complete machine advantageously will be readily understood. It is set on a corner of the land to be cleared and five stumps in the immediate vicinity are split, fired and placed under the blower blasts. These require no particular further attention until consumed. Second-growth trees are meantime pulled out and sawed into cordwood, the roots and snags being thrown into neighboring fires to be burned. With the power grubber all the undergrowth and small roots are removed.

Subsequently the machine is removed to another position and these operations repeated. By this method a strip of 50 to 100 ft. wide across the property is left ready for immediate plowing and seeding while the machine is engaged in clearing other sections. The noticeable value of the plant is in the expediency with which the work is executed, the reduced cost and the clearing of the land without waste of the vegetable mold which makes it so fertile and productive.

The stump burner is the design of Oscar T. Ross, of Portland, Ore., who was associated with C. F. Blake and E. J. Moore, of that city, in perfecting the entire plant.

SAMPLING COAL AT ST. LOUIS consists of driving a 2-in. pipe into a carload at several points in a car and taking the coal lodged in the pipe. Tested in six cars against the ordinary method of casting aside a shovelful at regular intervals and reducing by quartering the averages check within 1 per cent.

Tunnel Intake for Milwaukee Waterworks

A $2,500,000 Project Involving Twelve-Foot Tube, Built by Compressed-Air Methods, and a 120,000,000-Gallon Pumping Station

By J. A. MESIROFF
Formerly City Engineer of Milwaukee

IN spite of the fact that construction is being rushed on a project for increasing the water supply of Milwaukee by means of a new intake, lake tunnel and pumping station, it is expected that there will be a water shortage in the city before the new work is completed. The object of the new work is threefold—to increase the water capacity, to tide over periods of breakdown, and to secure a better quality of water by building an intake at a point further away from the mouth of the river than at present. This article describes the existing waterworks intake and pumping station, the new intake and pumping station and the connection between the two.

PRESENT SUPPLY

The present water supply consists of one 7½-ft. intake tunnel terminating in two shafts, one on shore and one 3146 ft. directly east from the center line of the shore shaft, the lake shaft connecting the tunnel with a crib. Two 5-ft. pipe lines extend from the crib directly east 5000 ft. and terminate in two submerged cribs. The submerged cribs are located on the lake bottom in a depth of 60 ft., the ends of the two pipe lines rising in the center of these submerged cribs to a height of 15 ft. above the bottom of the lake. The water supply is obtained 45 ft. below the surface and 8156 ft. from shore. On the shore the 7½-ft. tunnel terminates in a 10-ft. shore shaft. This shore shaft connects with the present pumping station through a 7½-ft. shore tunnel.

The pumping station now in operation has a capacity of 110,000,000 gal. in 24 hr. and is supplied through the 7½-ft. shore tunnel built parallel with the station and having three branch tunnels extending into the pump wells in the station. This pumping station is gradually being rebuilt and remodeled and when completed will be an up-to-date pumping station, having a capacity of 120,000,000 gal. per 24 hr. It will be practically as good as the new pumping station contemplated in the new project.

NEW SUPPLY

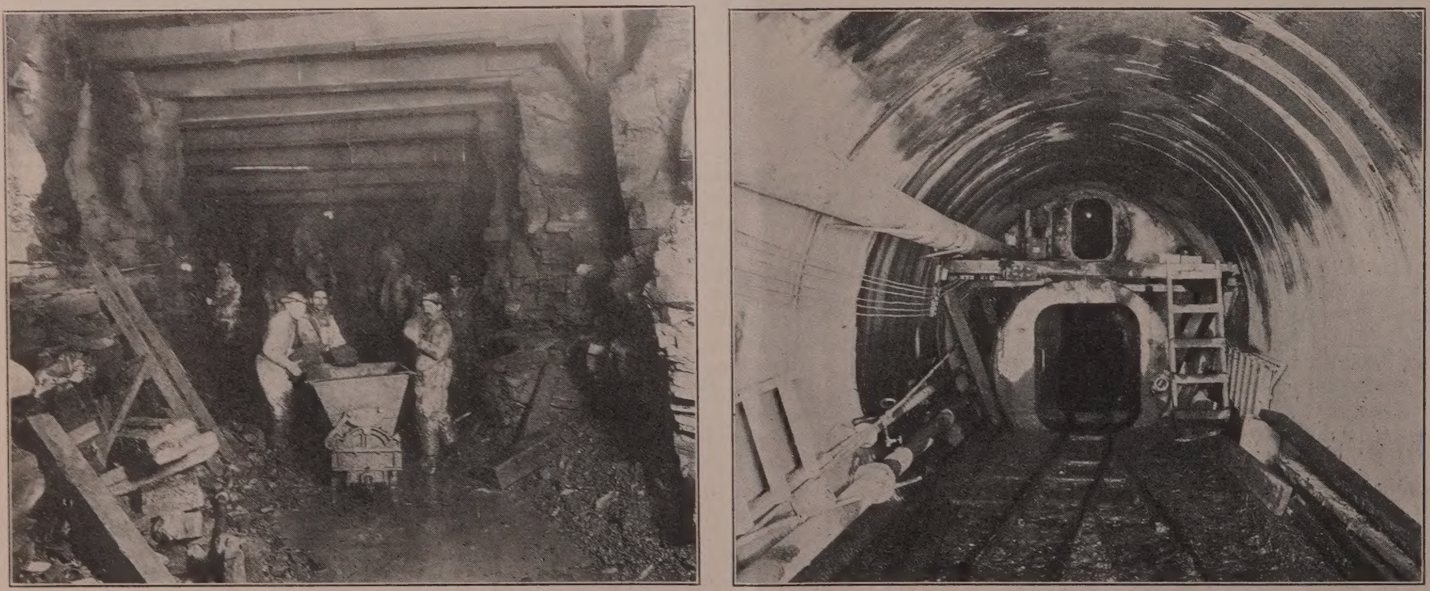
The new water supply which is now under construction includes one 12-ft. lake intake tunnel, terminating in two 15-ft. shafts, one on shore and the other in a lake shaft 4000 ft. out from the center of the shore shaft. In a northeasterly direction from the lake shaft pipe lines extend about 2500 ft. Each of these pipe lines terminates in a submerged crib located in 60 ft. of water.

Two 9-ft. tunnels will extend from the shore shaft, one along the shore to the present pumping station and the other directly west to the west shore of the Milwaukee River, connecting directly with the pump wells of the new pumping station. One pumping station, having a capacity of 120,000,000 gal. per 24 hours, will be constructed on the west side of the Milwaukee River.

The lake tunnel and shafts, for which a contract has been let, will cost approximately $450,000. The lake crib, it is estimated, will cost about $75,000. The estimated cost of the entire project is $2,500,000.

ORDER OF CONSTRUCTION

The order of construction decided upon is as follows: Lake tunnel and shafts, lake crib, shore tunnel, two 6-ft. pipe lines and submerged cribs, land tunnel, pumping station, and the remaining two 6-ft. pipe lines. In order to be able to use a part of the new project as soon as possible the lake tunnel and shafts were selected as the portion of the project to be built first, as it is considered the most hazardous portion of the project and is likely to cause delay. No portion of the project could be used without the lake tunnel. When the lake tunnel, shafts, crib and shore tunnel are completed the present pumping station can, in case



LOADING MUCK INTO CARS IN THE MILWAUKEE TUNNEL HEADING AND VIEW OF THE COMPRESSED-AIR LOCK of emergency, draw its supply of water from the new intake. When water is drawn from the new crib and the first two 6-ft. pipe lines and submerged cribs are completed the present pumping station will draw its supply from the new intake.

BORINGS

When the preliminary engineering and locations were decided upon borings were made to determine the strata along the line of construction. The borings made for the old intake were also used in order to better study the strata. It was found that over the lake bottom along the entire length of the tunnel there is a layer of clay of an average depth of about 5 ft. Below this was found a layer of hardpan consisting of cemented gravel. This layer is 16 ft. thick; underneath it were found black and brown shales interlaid with thin layers of soapstone.

Having found a good covering over the proposed tunnel, it was decided to build as close to the surface as possible to minimize the air pressure in the tunnel and a depth of 80 ft. was selected. This will keep the tunnel in brown shale and at the lake end the roof will be in black slate.

TUNNEL AND SHORE SHAFT

The contract for the construction of the 12-ft. lake tunnel and the two shafts was let on April 29, 1913, to Joseph Hanreddy & Company, of Chicago. The contract requires the work to be completed in two years. A great deal of trouble has been experienced in building the present intake, due to faults in the strata and also to the fact that the contractor did not have a plant to take care of water. A condition was inserted in the present specifications to the effect that the plant installed by the contractor for the construction of this work must be satisfactory to the commissioner of public works and the city engineer. This clause is considered advantageous, as the plant installed by the contractor will take care of any ordinary emergency. The compressors are operated by motors and the power has been continually kept on, even when no air pressure was used in the tunnel.

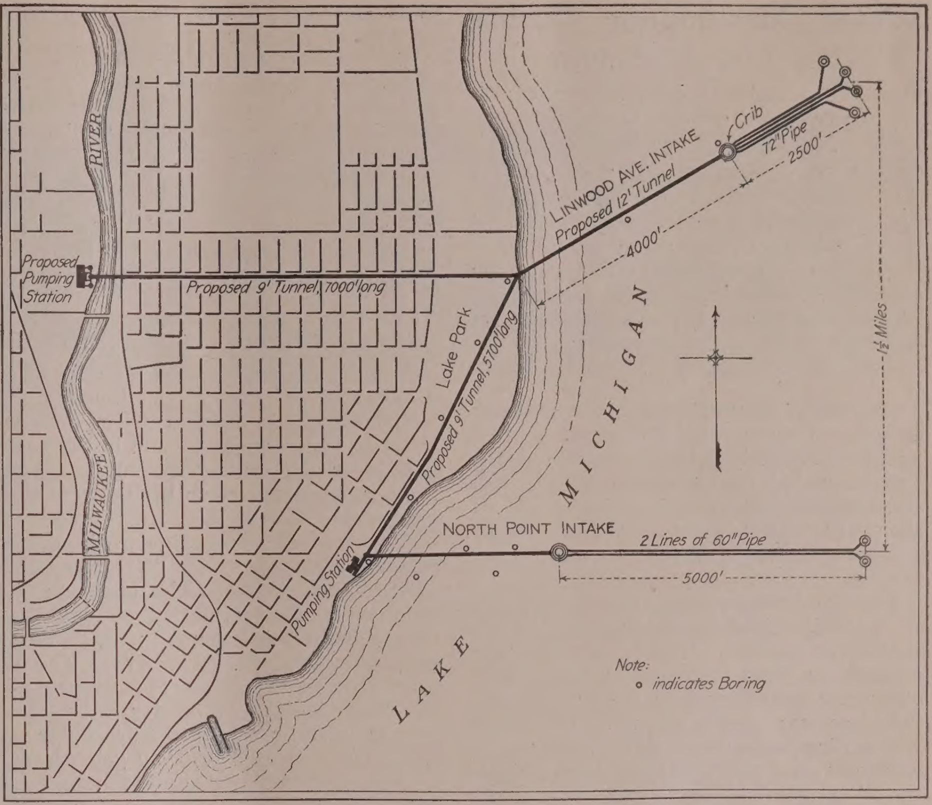
The center of the shore shaft was placed 25 ft. from the shore in about 3 ft. of water. Steel sheeting was used to start the excavation and no serious trouble was encountered during excavation. A hexagonal form of 12 x 12-in. timbers was used in timbering the shaft; these timber forms were placed 3 ft. apart. No concreting was done until the entire shaft was excavated.

The accompanying table shows the progress made on the shore shaft and tunnel:

PROGRESS ON SHORE SHAFT AND TUNNEL.

	Shaft		Tunnel	
	Excavation	Concrete	Excavation	Concrete
1913				
July	24	..	....	....
Aug.	88	..	134	....
Sept.	..	87	216	37
Oct.	..	..	216	151
Nov.	..	..	439	151
Dec.	..	..	578	414
1914				
Jan.	..	..	823	683
Feb.	..	..	1025	873
Mar. 17 .	..	..	1147	1018

Blaw steel forms were used by the contractor for the construction of the shore shaft and are now used for the concreting of the tunnel. Two 9-ft. openings, the bottoms of which are 20 ft. below city datum, are placed in the shaft for the two land tunnels.



PLAN OF PRESENT AND NEW INTAKE TUNNELS AND PUMPING STATIONS

The excavation of the lake tunnel was started on Aug. 14, 1913, and 230 ft. of the tunnel were excavated before any concreting was done. Because the shale or material along the line of the tunnel weathers and crumbles when exposed, it was difficult to keep the roof from falling. Numerous small falls have taken place up to the present time, but there has been none of a serious nature. However, the contractor was forced to keep the masonry work within 80 ft. of the heading. One of the photographs shows the heading of the tunnel at a point 1000 ft. from the center of the shore shaft. It illustrates the method of timbering used and gives an idea of the material encountered. It is contemplated, when the lake crib is completed, that part of the lake tunnel will be built from the crib and will meet the tunnel at a point about 2500 ft. from the shore shaft.

At a point 120 ft. from the shore shaft air locks were installed. The locks and plant are kept constantly in condition for emergency.

CHANGES IN CONSTRUCTION

Should there develop during construction any serious break or crack in the strata, allowing any great volume of water to flow into the tunnel, the method of lining will be changed to brick masonry instead of concrete, the contractors having bid on both brick masonry and concrete. The specifications provide that the city engineer may lessen the blasting charges if, in his opinion, the blasts are too heavy and dangerous for the covering. For this purpose a blasting chart is being kept daily by the inspectors. This chart illustrates the method of drilling, the depth of the holes, the size of holes and the amount of dynamite used per blast. These charts are scrutinized from time to time by a special inspector who is an expert on blasting. This course was adopted because of the limited amount of covering over the tunnel.

The material is removed from the tunnel with cars and mules, hoisted to the surface with elevators and then wasted. The concrete, timbering and all other material used in the construction of the tunnel are handled in this way. The excavated material is utilized by the City Park Board in providing protection to the park ground against the encroachments of the waves, which at this point break with great force. Protection piers have been constructed to retain the tunnel excavation and the vast quantity of spoil will afford park-shore filling for a considerable distance north and south of the tunnel.

INSPECTION

A very thorough system of inspection is being carried on in this work. The organization consists of one chief inspector and as many inspectors under him as are required to inspect the work properly. A report is being issued by the chief inspector daily. This report contains everything of importance—the amount of work done, the number of men employed, the amount and kind of material used, accidents, blasting charges, conditions of plant, weather conditions and such other information as may be of interest to the department.

VENTURI METERS used by the metropolitan water works, which supplies eighteen municipalities in the vicinity of Boston, numbered 71 during the year 1913. There are also three disc meters, one torrent meter and five detector meters of the Hersey type used in measuring small quantities which cannot be conveniently measured with meters of the Venturi type. The work of caring for the meters in the metropolitan district has been done by two men, assisted at times by a third. The cost of the work connected with the reading, operating and maintaining of these meters has been $1,974.38.

Water Consumption During the Fire at Salem

Francis F. Longley Records Results of His Observations at Scene of Conflagration June 25 That Swept 250 Acres

FOR the purpose of studying the water requirements of a city during a large fire, Francis F. Longley, of the firm of Hazen & Whipple, consulting engineers, of New York, visited Salem, Mass., a few days after the conflagration of June 25 and 26. The results of his investigations are embodied in the following report:

THE FIRE

The fire of June 25 and 26 burned over an area of more than 250 acres, approximately one-third of the area of the city. The estimated fire loss was $20,000,000, and the assessed valuation of the city is about $42,000,000. Only a few lives—about five—are known to have been lost during the fire.

The fire broke out at about 1.45 p. m. of June 25. Aided by a high wind, the fire spread very rapidly. Within three-quarters of an hour or an hour sparks had kindled other fires as far away as 1½ or 2 mi. and the blaze was beyond control. The entire Salem Fire Department was, of course, in service at once, and apparatus from numerous other cities was soon on the way. From such records as are available there were, in all, about 24 engines in service during the fire.

PUMPING

The Salem pumps were at once called upon for their maximum output, which was about 9,500,000 gal. per day, and with this quantity and the numerous engines at work at the fire, the pressure in Salem dropped off rapidly. At 3.30 p. m. a connection was made with a 16-in. main from the Peabody supply. This connection was on until 3 o'clock the following afternoon. The superintendent of the waterworks at Peabody estimated that water was delivered into the Salem mains from Peabody at a rate of 8,000,000 gal. per day. The pump record and reservoir record substantiated this estimate. At 8 p. m. a connection was made with the Danvers supply, which was kept on until 11 a. m. the following day. At about 9 or 10 p. m. of June 25 connections were made with the Beverly mains, which were on until 1 p. m. the following day. The estimated rate of flow from Danvers was about 4,700,000 gal. per day, and from Beverly, about 3,600,000 gal. per day. A connection was also made with a main from Marblehead, but was closed again within half an hour.

The service reservoir of Salem was shut off entirely during this period to enable the pumps to operate at a slightly higher pressure. The accompanying diagrams show the water quantities, the estimated aggregate rate of supply, the water pressure by a recording gage in the water department office and the estimated pressure above the same base at the Salem pumping station.

PRESSURES

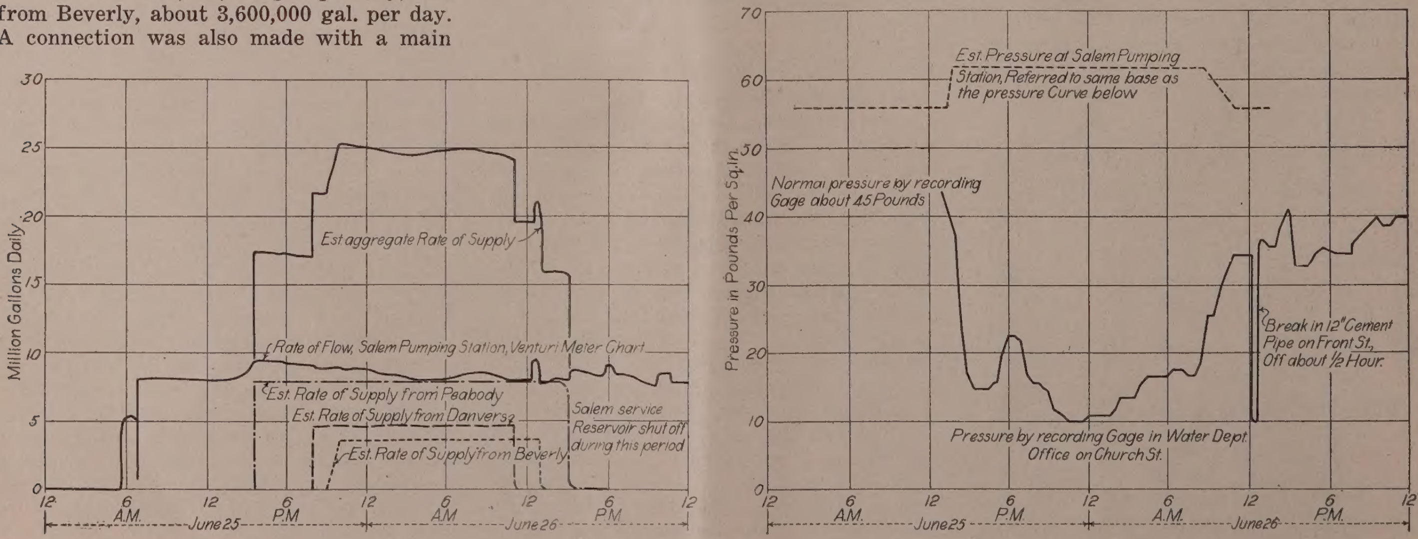
The normal working pressure in Salem, as measured at the water department office, is about 45 lb. Before the Danvers connection was made the pressure on the Danvers 8-in main was 93 lb. and in the Salem main near the connection, 20 lb. After opening, the pressure on the Danvers main dropped to about 50 lb.

The average consumption in the city of Salem is normally only little more than 4,000,000 gal. per day. The maximum rate, from the best information obtainable, is about 6,500,000 gal. per day; the minimum is perhaps 2,000,000 gal. per day. The diagram showing the estimated aggregate rate of supply indicates that for a period of more than 12 hr. during the fire water was used at a rate of 25,000,000 gal. per day.

There were no breaks in the mains to interfere with the use of the system during the height of the fire. After the fire was well under control and the pressure conditions were coming back to normal a break occurred in a 12-in. cement pipe on Front Street. This was promptly shut off and did not interfere in any way with fire protection. The diagrams, both of water quantity and pressure, show this break clearly.

QUANTITY INADEQUATE

The population of Salem at the last census was about 46,700; it is now probably about 50,000. By the conventional rule about 7,000,000 gal. per day should be sufficient for fire service. Adding to this the maximum rate previously stated, of 6,500,000 gal., gives a total of 13,500,000 gal. per day. The lesson that this seems to teach, Mr. Longley states, is that the quantity to be provided by that rule may not be more than about one-third to one-half, enough to take care of such a serious conflagration as the one which devastated the city of Salem.



CHARTS SHOWING QUANTITIES AND PRESSURES OF WATER DURING THE RECENT FIRE AT SALEM

Condition of Test Road after Two Years of Service

Report on Surface Treatments of Sections Built by Office of Public Roads at Chevy Chase, Maryland

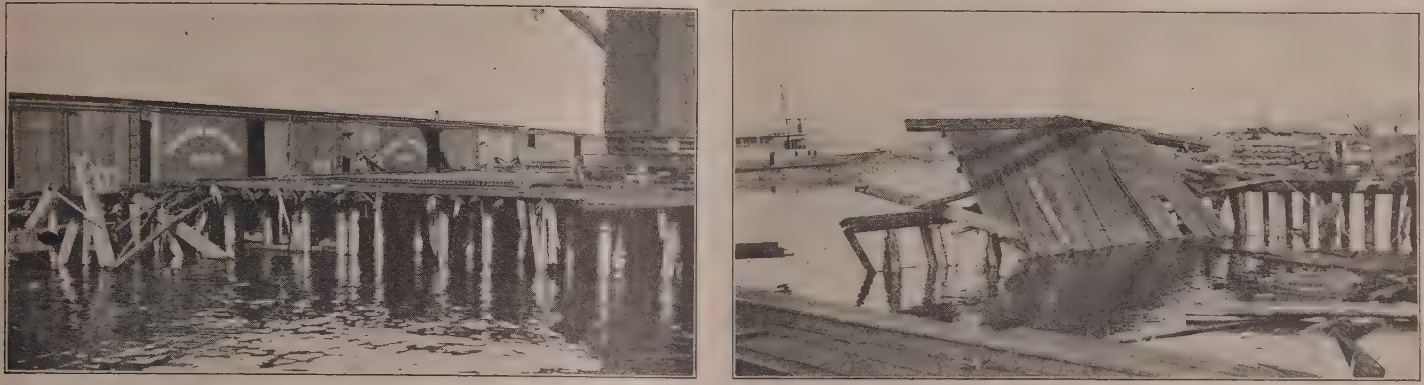
A PROGRESS report of experiments in dust prevention and road preservation on the 6195-ft. test pavement laid in 1912 at Chevy Chase, near Washington, D. C., forms part of Bulletin 105 of the U. S. Department of Agriculture. The pavement, which was built under the direction of the Office of Public Roads, is in six sections, of which five are bituminous or cement concrete and one is brick. The concrete sections—which are officially numbered 1, bituminous concrete (Topeka specifications); 2, bituminous concrete (District of Columbia specifications); 3, cement and oil-cement concrete surface treated with bituminous materials; 4, oil-cement concrete, and 5, cement concrete—were described in the Engineering Record of June 28, 1913, page 719. The vitrified-brick section was described on Sept. 27, page 347.

APPLICATION OF TREATMENT

Cold weather prevented application of the surface treatments until the spring of 1913, when the surface of the concrete was washed, thoroughly swept, and allowed to dry off, after which a number of bituminous materials, singly and in combination, were applied. Where a paint coat is indicated in the following discussion it signifies a thin surface application of a material for the purpose of possibly securing better adhesion than might otherwise be obtained for the wearing or carpet coat. The paint coats were broomed over the surface without heating. The heavier products were heated in 3-bbl. kettles and spread over the surface by means of a hand-drawn distributor of 30 gal. capacity, so designed as to permit the material to flow out in a thin sheet about 18 in. wide. The applications were then covered with pea gravel or limestone grits applied by a spreading machine and the road was immediately opened to traffic.

Expansion joints were purposely omitted and no joints of any other kind were constructed except those between the days' work, and these were formed at an angle of 80 deg. to the center line of the road.

Inspection of the sections on Jan. 29,



TWO VIEWS OF BREAK IN WHARF AT PORT ARTHUR, SHOWING SHARP LINE OF SHEAR AND FORCING UP OF DECK

1914, furnished the material for the report, which is here abstracted.

Experiments 1 and 2, the bituminous concretes, are both in excellent condition. The section laid under the Topeka specification has a slightly wavy surface as compared with the District of Columbia pavement, and in a few places has compressed below the top of the adjoining curb. The seal coat on the District of Columbia pavement began to bleed badly with the advent of warm weather, and in July it was necessary to make an application of pea gravel to the extent of 0.011 cu. yd. per square yard. A smooth and excellent wearing surface has resulted.

The results of expansion in concrete during hot weather were noted at the joints between experiments 2 and 3, where the roadway buckled across its entire width, and to a small extent sheared off some of the concrete base of the District of Columbia pavement. The defect was repaired by cutting out a narrow strip of concrete. The pavement settled to its original grade, and the slot was filled by three double courses of vitrified brick, the joints of which were filled with hot coal-tar pitch.

RÉSUMÉ OF BITUMINOUS TREATMENT

A summary of the conditions of the various sections at the time of inspection is included in the report. There seems to be no noticeable difference between the adaptability of cement concrete and oil-cement concrete to the treatment. Refined coal tar was used on two sections on each of which the adhesion was still fairly good, although the bitumen had become quite hard and had worn off in a number of small spots. A water-gas tar preparation was also used on two sections, with a resulting adhesion about as good as in the case of the coal tar, but with much fewer worn places.

Fluxed native asphalts containing about 95 per cent bitumen were used over two sections. On the first section the adhesion was not particularly good. There were no failures on the eastern part of the road, but for one-third of the width of the western side the treatment was about 30 per cent gone. The north half of the other section was good, but there were a few large bare places in the south half, on the east side of the road. This type of asphalt was also applied on another section over a water-gas tar preparation, and on still another over a native asphalt emulsion. For the first of the latter there were a few failures reported on the northern two-thirds of the section; the bitumen was flexible and the adhesion good. There was extensive patching in the southern third, but a large part of this was due to damage done by a traction engine shortly after the section was completed. Where the asphalt was used over the emulsion the adhesion was poor. There were a few small failures on the eastern half of the roadway and about 40 per cent of the treatment was gone on the western half.

Oil asphalts containing about 99.72 per cent of total bitumen were used on two sections, in one case alone and in the other applied over a water-gas tar preparation. Of the first stretch the report remarks that the section was generally good on the eastern half of the road, with several fairly large bare places along the western side. In the other the adhesion was poor and the treatment was about 50 per cent gone throughout. One very good piece, however, was left on the eastern half of the southern end.

Practically all of the sections suffered from the passage of a traction engine during a rather warm spell soon after the completion of the work, when the bitumen showed a slight tendency to bleed. The necessary repairs were rapidly made by the patrolman, and the sections were then permitted to wear without further attention.

OCCURRENCE OF CRACKS

The exposed concrete surfaces as a whole presented a smooth, uniform texture, and there was apparently no difference in wear between the plain-cement and oil-cement concrete. The addition of hydrated lime has had no noticeable effect on the concrete. This section cannot be distinguished from others and has developed four cracks in a total length of 260 ft. Transverse cracks developed at varying intervals throughout the sections, and the average distance between cracks is shown in the accompanying table, which is based on observations made Nov. 5, 1913. The cracks in the sections which had been surface-treated with bituminous materials include those which were noted before the surface treatment was applied as well as those which have since been noted through cracking of the bituminous surface.

It will be noted that cracks have developed at a great average distance where a limestone aggregate was used, and at the last inspection there were still two sections of the limestone concrete, each 140 ft. in length, without a crack. When the edges of any cracks began to wear excessively the cracks were filled with a water-gas tar preparation and sand. The results of this treatment have thus far proved entirely satisfactory.

AVERAGE DISTANCE BETWEEN CRACKS IN CHEVY CHASE CONCRETE ROAD

Surface coated with bituminous materials:	Aggregate	Length in ft.	Av. distance between cracks, in ft.
Cement	Gravel	660	45
Cement	Limestone	580	145
Oil-cement	Limestone	230	115
Oil-cement	Gravel	370	50
Surface exposed:			
Cement	Gravel	425	61
Cement	Limestone	600	100
Cement	Trap	300	40
Oil-cement	Limestone	210	105
Oil-cement	Gravel	260	45
Oil-cement	Trap	290	60

No difference was noted in the character of the various sections of brick pavement. The grouted surface had practically all worn off, and it was observed at a few places throughout the length of the sections that small transverse cracks had developed where the grout had broken loose from a course of brick.

Part of Wharf at Port Arthur, Ontario, Collapses

Load of Rails Causes Section of Wooden Structure about 75 Feet Square to Shear Off and Sink Out of Sight

By C. E. HENDERSON
Assistant City Engineer, Port Arthur

A SECTION of a wharf belonging to the Canadian Northern Railway sank out of sight July 4 under a load of about 2100 tons of steel rails. The "S. S. McKee" had completed unloading a cargo of rails in the afternoon and had left a short time before the accident occurred. The rails were 80-lb. A. S. C. E. rails, 33 ft. long, and had been placed in three piles, each pile being about 33 x 66 ft., with transverse spaces a few feet wide between piles. The section that failed was about 75 ft. square. Two piles of the rails were carried down with it, each pile containing about 1050 tons. The entire length of the wharf was loaded with rails, some of the piles weighing 1200 or more tons. The rails on the portion not wrecked were hastily loaded on cars to save from further loss and to facilitate the salvaging of lost rails.

The wharf, which was about 75 x 450 ft., was constructed in the winter of 1912 and 1913 by the Thunder Bay Harbor Improvement Company under the supervision of the Canadian Northern Railway according to plans and specifications furnished by the railroad. The pier was built similar to the others originally used for unloading steel rails and later was covered and used for package freight as occasion demanded. It is stated that some of these older wharves had supported piles of rails weighing as much as 1400 tons without sign of failure.

The piles used were sound tamarack from 40 to 50 ft. long, with 12-in. butts, and were driven to hard bottom. The water varied from 12 to 20 ft. in depth. In places the bottom of the lake was covered with several feet of soft mud. No batter piles or sway bracing was used. The piles were spaced about 5 ft. each way and were capped with 12 x 12-in. fir running transversely. The longitudinal stringers were 6 x 10-in. fir spaced 24 in. on centers. The flooring was 2 x 12-in. material. The deck is about 8 ft. above the level of the lake. The portion of the wharf which remained standing supported several piles of rails, the edge of one being only a few inches from the rupture.

VIEWS ON PRECEDING PAGE

The left-hand illustration shows the east end of the break and the portion which has supported several piles of rail. It will be noted that the stringers were sheared off on a straight line across the wharf. The few rails shown are some of those which had been removed by derrick and diver. The other view shows the slip side and west end of the break, also the portion (about 12 ft.) of wharf which remained standing at that end. It shows the manner in which a section on the slip side was shoved upward. A section on the track side was shoved upward in a similar manner. These views would indicate that the first part to go was the east end of the break where the stringers were sheared off. This is proved by the manner in which the rails are lying in the water. The diver who is removing the rails states that they have slid toward the east end of the break. The cause of the accident will probably not be known until the wreckage is cleared away so the condition of the piling underneath can be investigated. If the rails were properly piled with a skid rail over each cap the piling would have been supporting about 11 tons each.

Industrial Track Equipment for Concrete Road Building

Adaptability of Narrow Gage Plant for Handling Materials and Cost Data on Work for Illinois Highway Commission

HOW 2-ft. gage industrial railway equipment, consisting of portable tracks, double side steel dump-cars and locomotives, may be adapted to the economical handling of material for concrete road construction was explained at the National Conference on Concrete Road Building last February in the discussion of the report of committee 10 by Louis Hellmann, who is manager of the Orenstein-Arthur Koppel Company, of Chicago. Mr. Hellmann's brief for the industrial track system is given below:

Few people realize, when suggesting this means of transportation on road-building jobs, how easily a special outfit of this kind is installed, and how cheaply it operates. Consider, therefore, that the track on which the cars and locomotive operate is a built-up unit, including steel ties to which the rails are fastened, and including angle joints all ready to be laid down. Two men are able to handle a section, which comes in standard 15-ft. lengths, and weighs 225 lb. per section for 16-lb. rail, and 270 lb. per section for 20-lb. rail, with preference for the latter. You can figure that the laying of the track, consisting in connecting of the different sections, will not exceed ½ cent per foot of track, or about $26 per mile, where it is laid from the point of arrival of the material.

Where the track is to be hauled across the country for a distance, or where, after being used on one job, it has to be transferred to another one 3 or 4 miles away, you can reckon on 1 cent per foot, or about $53 per mile as the average cost for such hauling, laying and connecting at the new location. The figures for laying, hauling and connecting of the track are prices at which this work has been sublet many a time, and represent no guesswork, but true, actual cost. Switches likewise are one unit of the same length as a section of track, so that where turnouts are to be made any straight section can easily be replaced by a switch or curve of the same length.

TYPE OF CAR

The V-shaped cars used in connection with this track are of the double side steel type, 1½-yd. capacity, weighing about 1000 lb. each. There is no danger of the contents of the car when dumped rolling or falling back on to the track and interfering with the operation of the train. The car clears itself entirely, the center of the dumped load being about 3 ft. from the edge of the rail. An advantage also worth mentioning is that there is no guesswork about the contents of each load as there is in the case of wagons, because you can be sure that each car always holds the same amount of material. These cars are used not only for hauling sand, gravel or crushed stone, according to the specifications of the road, but are also used for handling the cement, one car generally carrying about 10 barrels. I have also heard of instances where water was carried along in the train in a car easily designed for this purpose.

For hauls shorter than ½ mile, a team of horses hauling seven loaded cars will be economical, whereas on hauls exceeding this distance, the preference will have to be given to locomotive haulage. The locomotives most widely used are a 20-hp or a 30-hp engine, weighing 13,000 and 15,000 lb. respectively. As astounding as it may sound, I have witnessed the 30-hp engine pull twenty-six loaded 1½-yd. cars up a 5 per cent grade, not as a record performance, but as an hourly duty for weeks. This seemed to me to be so remarkable that I checked up the grade and then secured a profile from the county engineer, which verified the above figure. For steeper grades, say 6 per cent to 10 per cent, a geared engine is used to good advantage. While somewhat slower and considerably heavier (just about double the weight) it still can operate with perfect safety on a 20-lb. track, the weight being distributed over eight wheels instead of four.

INDEPENDENCE OF WEATHER

This system, as a whole, has been found to have the advantage of working under almost any weather conditions, thus enabling the piling up of the material alongside of the road even in times when the weather does not permit of actual work. It will work with an average speed of between 8 and 12 miles per hour, therefore covering much larger mileage per day than would otherwise be possible, and at a cost that will be materially below the cost of hauling by other means. This system has, therefore, strong advantages in the points where the others are weak — independence from weather conditions and speed. The track can be laid over any improved road without the least fear of damage. The outfit also requires less space for maneuvering than is required by a team or traction outfit, as the overall length of the ties is only 32 in.

There is a Milwaukee County concrete road where the material came in from both sides of the road to be built, necessitating the hauling of the material on the berm, because it was, of course, impossible to haul over the green concrete. The space available was too small for either a team or traction outfit or motor truck, and this track, car and locomotive system was really the only solution in this case. The locomotive does not need to be turned around, as it pushes the cars just as well as it pulls them, and both the 20 and the 30-hp engines are able to go through one of the 15-ft. switches which have a radius of 30 ft.

Besides being used for the hauling of road building material this same plant can be used for any earth moving proposition. Where there are cuts or fills to be made, the cars can be put to work under a steam shovel.

The cars can be held in a semi-dumped position, and in this position filled to three-quarters of the capacity, the height in semi-dumped position being 30 in. and the total height 46 in.

COST DATA

As to actual cost data on hauling with this outfit a few figures are compiled from actual work done last year in Sargent and Embarrass Townships under the supervision of the Illinois State Highway Commission. In this case the townships rented a complete outfit consisting of over 6 miles of track, some cars and locomotives. The State guaranteed to the commissioners that the cost of hauling on the longest haul, which was over 6 miles, would not exceed 63½ cents per cubic yard for the entire distance, or slightly over 10 cents per cubic-yard-mile. This figure is the cost per cubic-yard-mile, and not per ton mile, the cost per ton mile, of course, being proportionately less. Upon completion of the job it was found that the actual cost did not exceed 54 cents, or 9 cents per yard-mile.

The figures were on the basis of renting the outfit, and include besides the rent, the freight to and from destination, the cost of laying the track, the cost of operating the equipment, the proportionate expenses of a superintendent, in fact everything that can be charged up under the item "hauling costs." Had the outfit been owned instead of rented, these figures would, without doubt, be more favorable, because the depreciation figured against one year's work as a rule will not exceed 15 to 20 per cent of its cost, whereas the rental figures are of necessity higher than such depreciation. Besides, in case of ownership, the item of return freight need not be considered.

Contractors are accustomed to figure on team haulage 25 cents per yard on the first mile, and 20 cents per yard on each additional mile. To give a comparison of this cost if done by track, cars and locomotives, a figure of 8 to 9 cents per yard-mile is amply safe. Contractors using this outfit state that their cost for hauling after the track is laid has been 10 cents per cubic yard for a distance of 5 miles, or 2 cents per cubic-yard-mile. This figure of 2 cents has to be understood of course only for the operating of the outfit and the dumping of the material. If another 2 cents is added for the laying of the track, and another 4 cents as a maximum for depreciation, the figure of 8 cents to 9 cents per yard mile, as arrived at on the Illinois job, is verified.

Deformation and Deflection in Concrete Beams

New Formula Whereby Deflection Can Be Calculated as Soon as the Loading and Size of Beams Are Known

THE unit deformations in the steel and the extreme fiber of the concrete in beams are the only determining factors in deflection, except depth, span and method of load distribution, according to a paper read by G. A. Maney at the seventeenth annual convention of the American Society for Testing Materials held at Atlantic City June 30 to July 3.

A formula has been derived and verified by reliable tests, by which the deflection is equal to the product of a constant into the sum of the deformations at the two extreme fibers. The formula is based upon a certain property of the *M/EI* curve for any beam, namely, that the deflection of the beam is proportional to the statical moment of the area under the *M/EI* curve. This curve is obtained by dividing each ordinate of the bending-moment curve by the product of the corresponding values of *E* and *I*, which are the modulus of elasticity of the material and the moment of inertia of the sectional area of the beam, respectively. The statical moment of the area under the *M/EI* curve between any two points on the elastic curve, taken about one of these points, is equal to the vertical deflection of this point from a tangent to the elastic curve at the other point. (The beam is assumed to be horizontal.)

PROOF OF THEORY

The unit stress on the extreme fiber is equal to Mc/I. The unit deformation, e, of that fiber is therefore equal to Mc/EI, and the total deformation in the distance dS along the elastic curve equals $McdS/EI$. Referring to Fig. 1, it is seen that this total deformation of the extreme fiber, divided by the distance c from that fiber to the neutral axis, is equal to the change of slope, $d\theta$, in the distance dS. Or

$$d\theta = \frac{Mc\,dS}{EI} \div c = \frac{M\,dS}{EI} \qquad (1)$$

It will be seen that the term $M\,dS/EI$ is equal to the area under the *M/EI* curve for the distance dS along it; that is, $d\theta$ is equal to a differential element of the area under that curve.

Now, let S equal the distance from any differential element dS to the point at which the deflection is desired. Then, referring to Fig. 1, it is clear that the deflection of N' from the tangent MN at M is equal to the sum of the products of each $Sd\theta$ along the elastic curve from M to N, S being measured from N'. Or

$$f = \Sigma\, S\, d\theta \qquad (2)$$

But $S\,d\theta$ is equal to the statical moment of the differential element of the area under the *M/EI* curve about N', by the definition of statical moment. Therefore, the deflection of any point from the tangent to the elastic curve at any other point is equal to the statical moment of the area under the *M/EI* curve about the first point. (In this proof, as in all treatments of the elastic theory, it is assumed that the projection of the neutral axis after bending, upon its original position, is the same as its length before bending.)

This property for homogeneous beams may be expressed:

$$f = kl^2\,(M/EI) \qquad (3)$$

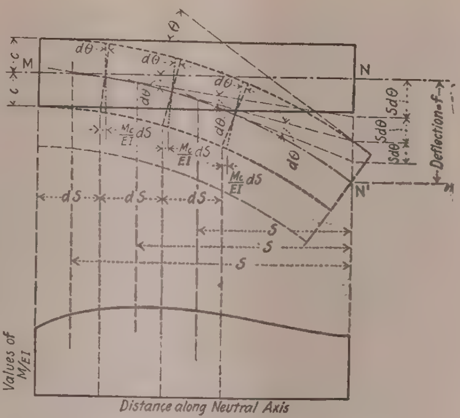


FIG. 1—M/EI—CURVE FOR BEAM

The value for *M/EI* in equation (3) will be taken for the point in the beam at which the bending moment is maximum. Then the value of k is such that the statical moment of the *M/EI* area between any two points, which is equal to the deflection of one point with respect to the tangent to the elastic curve at the other, is expressed in terms of this maximum value of *M/EI* and the span of the beam, l.

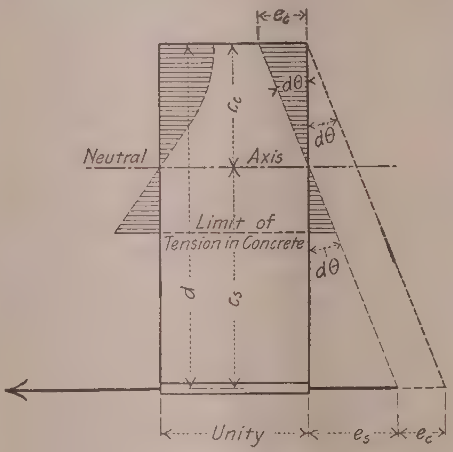


FIG. 2—VARIATION OF STRESS AND DEFORMATION

It is evident that the value of k for any case is equal to the coefficient of deflection for homogeneous beams in the well-known equation $f = k_1\,W\,l^3/EI$, divided by the coefficient for the maximum bending moment, which in general may be expressed as $M = k_2Wl$, from which $W = M/k_2\,l$. Substituting this in the last equation for deflection,

$$f = k_1 \cdot \frac{M}{k_2 l} \cdot \frac{l^3}{EI} = \frac{k_1}{k_2}\; l^2 \left(\frac{M}{EI}\right) \qquad (4)$$

Comparing equations (4) and (3), it is seen that $k = k_1/k_2$. To illustrate, consider the deflection at the center of a freely supported beam with a single concentrated load at the center; that is, the deflection of the support with respect to the tangent to the elastic curve at the center. The coefficient k_1 is 1/48, and k_2 is 1/4. Therefore the coefficient k in equation (3) will become $1/12 = 0.0833$ (see Fig. 3b).

The maximum deflection of a simple beam is equal to the deflection of the support from the horizontal tangent to the elastic curve. Usually the maximum deflection occurs at the center of the beam, as in all cases of symmetrical loading. With irregular loading, it usually occurs near the center of the beam. Then, by the property of the *M/EI* curve above referred to, we may express this deflection as the statical moment about the support of the area under the *M/EI* curve between the point at or near the center of the span, where the slope is zero, and the support. If this statical moment is expressed in terms of the maximum ordinate of the *M/EI* curve and the whole span, an expression is obtained of the form of equation (3), in which the coefficient k will equal k_1/k_2.

It may be of interest here to point out that *M/EI* equals the rate of change of the slope of the elastic curve, or the change of slope per unit distance. This may be seen by dividing equation (1) by dS, which gives $d\theta/dS = M/EI$.

DEFLECTION OF REINFORCED-CONCRETE BEAMS

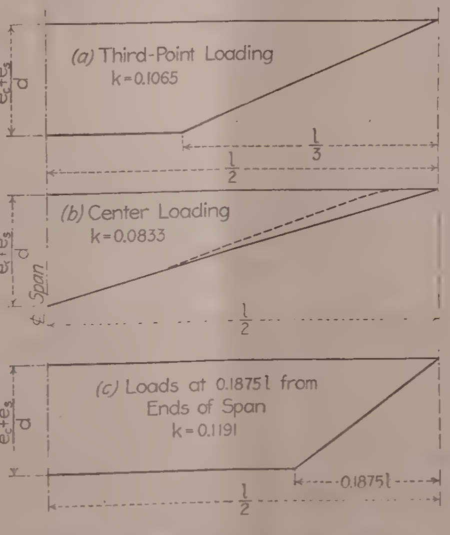


FIG. 3—VARIATION IN VALUES OF $(E_c + E_s)/D$

The application of this principle to the deflection of reinforced-concrete beams consists in finding a convenient expression for the value of *M/EI*.

Let e_c and c_c be the unit deformation in the extreme fiber and the distance from the extreme fiber to the neutral axis, respectively, for the concrete, and let e_s and c_s be the same quantities for the steel. In a homogeneous beam, as above indicated, $e/c = M/EI = d\theta$, and is the same for both extreme fibers. In this case, since the values of EI are constant throughout the length of the beam, e/c varies directly as M throughout the span.

In a reinforced-concrete beam (see Fig. 2),

$$\frac{M}{EI} = d\theta = \frac{e_c}{c_c} = \frac{e_s}{c_s} = \frac{e_c + e_s}{d} \qquad (5)$$

Substituting this value of *M/EI* in equation (3), the following expression for the deflection of a reinforced-concrete beam is obtained:

$$f = k\frac{l^2}{d}\,(e_c + e_s) \qquad (6)$$

This is based on the usual assumption that a plane before bending remains a plane after bending, which seems justifiable from the evidence of reliable tests herein referred to.

From the preceding analysis it is evident that the deformations of the extreme fibers are the only determining factors in the deflection, except the span, depth of beam, and load-distribution. It is also evident that the distribution of the stresses in the steel and concrete over the section has no influence on the deflection, except in so far as the stresses influence the deformations of the extreme fibers.

INFLUENCE OF TENSION

The influence of tension in the concrete might well be discussed here. From the principles of equilibrium it is known (see Fig. 2) that the effect of the tension in the concrete at low stresses is to reduce to some extent the compressive stresses in the concrete and the tensile stresses in the steel. This means that a stiffer beam might be expected in the earlier stages of the loading, as the steeper slope of deflection and deformation curves show.

For the reason, therefore, that tension exists in the concrete and that usually near the end of the beam the rods are bent up, the value of the $(e_c + e_s)/d$ curve, from which the deflection is obtained, will not have the same variation as the values of M.

Near the supports of a simple beam and at all points where the bending moment is small, we would expect considerably smaller values of $(e_c + e_s)/d$ than the value of M at such a point, relative to the value of M at the point of maximum moment would indicate. The values of $(e_c + e_s)/d$ would probably, follow the dotted line indicated in Fig. 3 b, because of the tension in the concrete. A glance at Fig. 3 b will show, and computations will prove that a small decrease in values of $(e_c + e_s)/d$ $(= d\theta)$ near the point about which moments are taken (the support, in this case), changes the value of this moment only slightly.

Values of k for maximum deflections under several conditions of loading are here given:

Beam with uniform load:

Ends freely supported, $k = 5/48$ or 0.1041;

Ends fixed, $k = 1/32$ or 0.0313.

Beam loaded at the third points:

Ends freely supported, $k = 23/216$ or 0.1065;

Ends fixed, $k = 5/144$ or 0.0347.

Beam loaded at the middle:

Ends freely supported, $k = 1/12$ or 0.0833;

Ends fixed, $k = 1/24$ or 0.0416.

Cost of Water in Milwaukee

WATER cost in Milwaukee per 1000 gal. during 1913, based on the total amount pumped and the total ordinary expenses, was 1.709 cents. Based on 80.57 per cent of the total pumped, for which the department received revenue, the cost was 2.12 cents. On the total pumped and all expense, including sinking-fund charges, depreciation, taxes and 4 per cent interest on net invested capital, the cost was 4.46 cents. The revenue, based on the total gallons pumped, was 5 cents per 1000 gal. Items of taxes and interest are not actually charged on the books, as the department has used these moneys for new construction since 1896 without the necessity of issuing bonds. A recent bond issue of $300,000 was made necessary on account of a charter provision requiring the city to have a balance exceeding any contract chargeable. The new intake tunnel amounts to $450,000, but as the work extends over a two-year period the finances of the department would have made provision for it out of its earnings.

TYPICAL INTERIOR; STIFFENED COLUMNS AND ROOF TRUSS SWAY BRACING

Two-Blast-Furnace Plant at West Duluth

Latest Designs in Buildings, Arrangement and Equipment Are Included in This Plant, Built at the Head of Lake Navigation

CONCRETE-BLOCK buildings, almost complete electrical drive, provision for indefinite extensions, a highly diversified mill and utilization of offheat from its open-hearth furnaces in boilers are some of the features in the new two-blast furnace plant of the Minnesota Steel Company at West Duluth, Minn. One of the factors making possible the location of the plant at the head of the lakes is the comparatively recent advance made in recovering by-products from the coke ovens, which are supplied with coal brought to Duluth in returning ore vessels.

The site is within Duluth City limits, 9 miles from the business section. It is reached by the Northern Pacific Railroad, but most of the raw materials will be received over the newly constructed Spirit Lake Transfer Railroad, 18½ miles long. Connections are made with the Duluth, Missabe & Northern, over which the ore will be received continuously from the iron range. The Spirit Lake line runs to the coal docks at Duluth. Outgoing material will be routed over the new Interstate Bridge via the Interstate Transfer Railway to Allouez Bay through Superior, where connections are made with all roads entering Duluth, except the Duluth & Iron Range. As navigation is open only from June to December, coal, limestone and dolomite must be stored for the remainder of the year.

PLANT LAYOUT

As laid out at present, the plant comprises two blast furnaces, ninety coke ovens of 1500 tons capacity, ten 90-ton open-hearth furnaces, a 40-in. blooming mill, 28-in. finishing mill, a merchant mill, a 40,000,000-gal. pumping station, a power station housing gas-engine-driven generators and blowers, a machine shop, hospital and incidental buildings. A concrete plant, the Ferguson concrete-block machine and a method of handling concrete in foundation work were described in the Engineering Record in the issue of Nov. 5, 1910. Due to the topography, the main longitudinal axes of the buildings are laid out on base lines, making an angle to the northeast of 37½ deg., with a true north line. Although the buildings are grouped, as shown in the layout herewith, they are sufficiently far apart, particularly longitudinally, for indefinite extension. The open-hearth-furnace group is built on a skew, which eliminates one reverse curve and brings the north end nearly opposite the rolling-mill building. Ultimately another open-hearth group will occupy a symmetrical position on the other side of the rolling mill.

The power plant is located between the blast furnace and the rolling mill, where so much of the power is used. The storage pit is entirely outside of the building site and so located as not to interfere with trains hauling directly to the plant. Water comes from the lake about ½ mile from the plant and is delivered through two lines, one on each side of the plant, with a cross-connection at the upper end of the open-hearth buildings.

Each blast furnace, which is of the thin-shell type, is equipped with five instead of the usual four stoves. Each stove is 22½ ft. in diameter and 101 ft. high and contains 400,000 corrugated checker brick. About 20,000,000 firebrick will be required throughout the plant. A stack 100 ft. high is placed on top of each set of stoves. Gases from the blast furnaces collected under the Neeland top are passed through dust catchers, grid and Ernst washers before delivery to the 150,000-cu. ft. gas tank at the end of the power house.

OPEN-HEARTH FURNACES

With the exception of one trestle approach to the open-hearth plant practically no trestle work is in evidence at this point. Molten metal is received from the blast furnaces in 60-ton hot-ladle cars on the ground floor and raised by a 90-ton crane to one of two 600-ton mixers, from which it is transferred by a 100-ton crane in ladles to the open-hearth furnace. The cold material is charged from a concrete floor 18 ft. above the ground level. Ingots are transported by locomotives to the storage yard or direct to the rolling mill.

Waste and offheat from the open-hearth furnaces are utilized to raise steam in ten Rust 437-hp vertical tube boilers, through which gases from the furnaces pass on the way to the stacks.

Four Babcock & Wilcox boilers of 400-hp capacity each are the only coal-fired boilers in the plant. Steam from these units is used principally in the engine operating the 40-in. blooming mill and in coke ovens.

The open-hearth furnaces are of the stationary type, with water-cooled Knox posts and buckstays, 16 x 40 ft. in plan. Seven have been built and three more are in course of construction. Fourteen will be the ultimate number.

FURNACE FEATURES

The features of this plant are the operation of the pouring spouts by a monorail 10-ton trolley hoist, motor-operated gates and a concrete charging floor. A 5-in. floor slab is laid in sections 4 ft. square on jack arches of concrete, filled over expanded metal sprung between the lower flanges of the steel I-beams, which vary in spacing from 18 in. up. The ½-in. joints are poured with a bituminous filler. Concrete was chosen rather than the usual brick or steel-plate floor to minimize blistering the men's feet and to present a smooth surface from which to shovel.

These furnaces are housed in a steel-frame building with 13-in. curtain walls of concrete blocks. The column bays are spaced on 28-ft. centers. Blocks were chosen because the material was on the ground and because the minimum amount of labor was required to lay them. The building for the ten furnaces is 984 ft. long, 140 ft. wide and 63½ ft. high from the ground to the bottom of the roof trusses. The entire width of the group, including the mixer, ore bins, producer building and stockhouse, is in the neighborhood of 340 ft.

REINFORCED-CONCRETE DOORS

The furnaces occupy a central position, with a craneway on each side. The central row of columns supporting the ends of the two crane girders is 7 ft. off the center of the building.

STRUCTURAL STEEL DESIGN

The structural steel was designed and fabricated at the various plants of the U. S. Steel Corporation.

In general the designs follow closely the desirable features set forth by Robbins Fleming, designing engineer for the American Bridge Company, in three issues of the Engineering Record, beginning in the issue of Oct. 25, 1913, page 467. Usual loadings for snow and wind pressure were allowed. For members subject to moving loads a factor of safety of 10 was used, while 5 was used for other girders, with the load consisting of dead, live and 20 per cent of live for impact.

At the north end of the open-hearth building, where extensions have been started, corrugated galvanized-iron, instead of the blocks, has been placed. The Fenestra sash are glazed with wire glass wherever there is any danger of injury from flying glass. Some of the panes are ⅜ in. thick in buildings where molten metal may spatter against the glass. Sills and lintels of windows are cast in one piece and the special sides in sections.

Ventilation is attained through the monitors, which are 12 ft. off the main roof and have sash 7 ft. high, every other one sliding.

Safety walks with railings are built over the lower chord of the roof trusses and also on the outside of the building at the height of the cranes. These and all other safety measures throughout the plant were installed after much discussion with the central safety committee of the corporation. The walk is carried from truss to truss, or on brackets on the columns, by the latticed sides. The walk is composed of 20 x 28-in. reinforced-concrete slabs 2 in. thick.

Car-entrance doors are usually of the steel lifting type, but all others are 10 ft. high and of two leaves, each 5 ft. wide, built in the machine shop. Each leaf is a structural frame with concrete plastered on Ferroinclave, with expanded metal wrapped and tied around the steel shapes.

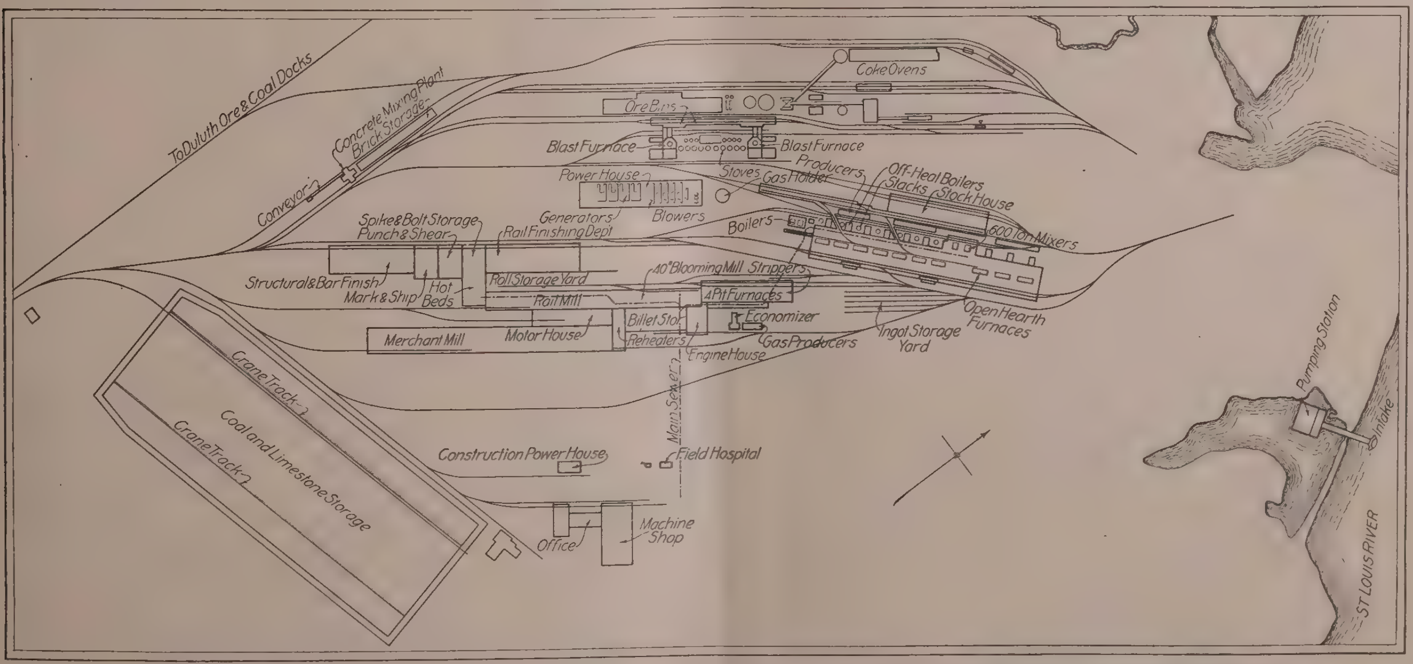
All stacks for the open-hearth furnaces and various producer plants are of reinforced concrete, built by the General Concrete Construction Company, of Chicago. Eight were built in ten weeks by the use of sliding steel forms. Universal Portland cement is used throughout in all of the mass concrete and for making the blocks.

POWER HOUSE

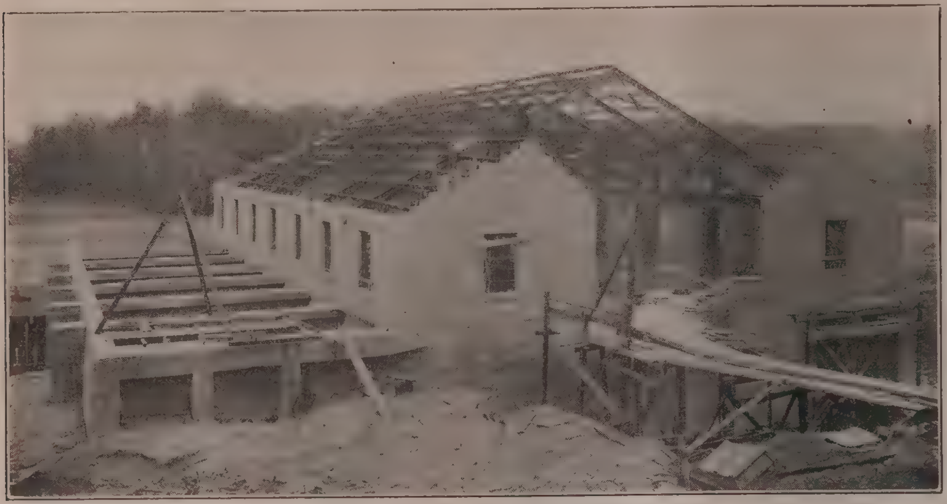
The power house is 528 ft. long, 110 ft. wide and is served by a 50-ton crane. All cranes, of which there are thirty, ranging from 10 to 150 tons capacity, are furnished by the Morgan Engineering Company. In this building are housed five gas-driven Bethlehem blowers of 20,000 to 28,000-cu. ft.-per-minute capacity, four Allis-Chalmers gas-engine driven Westinghouse generators of 3000-kw capacity, one 2000-cu. ft.-per-minute air compressor and three 750-kw rotary converters.

The building is of the same type as those described except for the Federal concrete roofing tiles, which are also used for the machine shop and motor house.

A concrete conduit on the blast-furnace



GENERAL LAYOUT OF PLANT AT WEST DULUTH, SHOWING SPACE BETWEEN BUILDINGS FOR FUTURE EXPANSION



CONCRETE-BLOCK PUMP HOUSE UNDER CONSTRUCTION, WITH FOREBAYS ON LEFT

side of the power house carries the pipes for gas and provides inlets to ventilate the basement, which is unusually deep, to insure ventilation, the lack of which might prove serious in case of gas leaks.

Rolling-Mill Buildings

The buildings extend for nearly 2000 ft. along the longitudinal axis of the rolling-mill group, of which the merchant mill is the longest structure. It is 1040 ft. long and 112 ft. wide. The maximum width of the group is 467 ft. With the exception of the motor building, which is entirely surrounded by concrete-block curtain walls, there are few partitions. Arrangements in these buildings are best described by following the metal from ingot to finished product. Entering at the northern end, the ingots are stripped by a 150-ton motor-driven stripper and then either placed in storage or dropped by one of the two 7½-ton charging cranes into one of the four soaking pits, each of which has four holes covered by an electrically operated door. These motors and the gas valves are operated from a gallery.

The producers are located 70 ft. to the east, in a separate building. Waste gas from the pit furnaces passes through an economizer to heat boiler-feed water from exhaust steam returning from the blooming mill engine to the open-hearth offheat boilers.

Removed from the soaking pits, the ingots pass to the 40-in. blooming mill operated by the only large steam unit in the plant. It is a 40 x 60 x 60-in. twin tandem Allis-Chalmers engine housed in a separate 86½ x 120-ft. building.

Leaving the blooming mill the billets are carried forward to a new type of up-cut Morgan 10 x 10-in. shears. From here the billets may enter storage to the left or may pass by a rotary transfer straight ahead to the rail mill, or to the left to reheating furnaces supplied from four producers.

Billets for the rail mill may or may not be reheated here. They pass first to a combination 28-in. rail mill, where structural shapes, all sizes of rails, splice bars, flats, rounds and squares are produced. This mill comprises eight stands of rolls, the first five passes of which are through two-high stands, the sixth to ninth, three high, and the tenth, two high.

These stands are operated by one 6000-hp Westinghouse motor through a bevel-geared line shaft 300 ft. long and 20 in. in diameter for nearly its whole length.

From the finishing roll the products pass five sliding hot saws, which may be operated separately or in gangs; then by a camber machine to a tip-up machine for beams, just before entering upon the hot beds, across which the material is propelled transversely toward the finishing and storage departments. The rail-finishing department is equipped with eight straighteners, sixteen drill presses and a cold saw for resawing seconds.

Merchant Mill and Machine Shop

Reheated billets are passed to the 16-in. nine-roll continuous mill, rope driven by a 3000-hp motor, thence to a 12-in. three-roll train direct-driven from the above motor. Such stock as must pass smaller rolls is returned to 10 and 8-in trains operated by 800 and 650-hp motors respectively, the location of these rolls being between the 16 and 12-in. mills. Leaving the last roll the stock passes to a Morgan cooling flat bed, 300 ft. long, then to shears and to storage and shipping space at the end of the building, served by two 10-ton cranes.

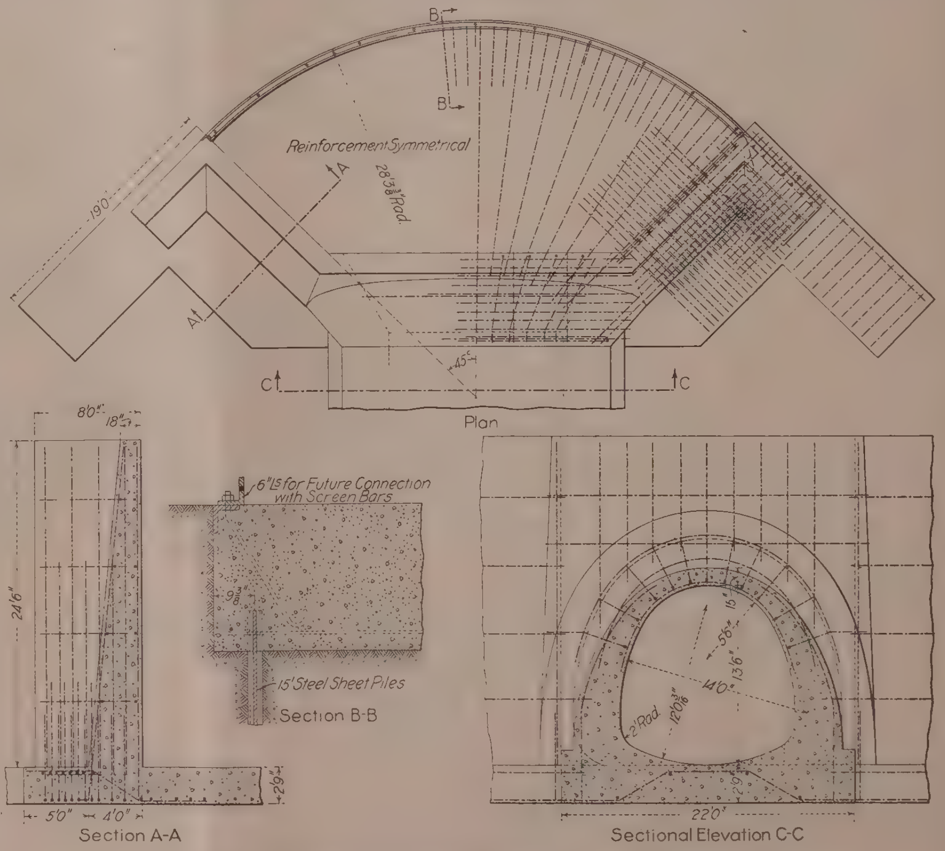
Special attention has been given in the 140 x 280-ft. machine shop to provide a maximum amount of light and to eliminate dangerous belting and wiring. The building is of the same general type as the others and has a concrete-tile roof with glass inserts. In the side walls Fenestra triple-size steel sash, 13 ft. wide by 12 ft. high, occupy a large percentage of the wall area.

All machines are motor operated, with the wiring here, as in all buildings, passed through fiber conduits embedded in the concrete floors or substructures.

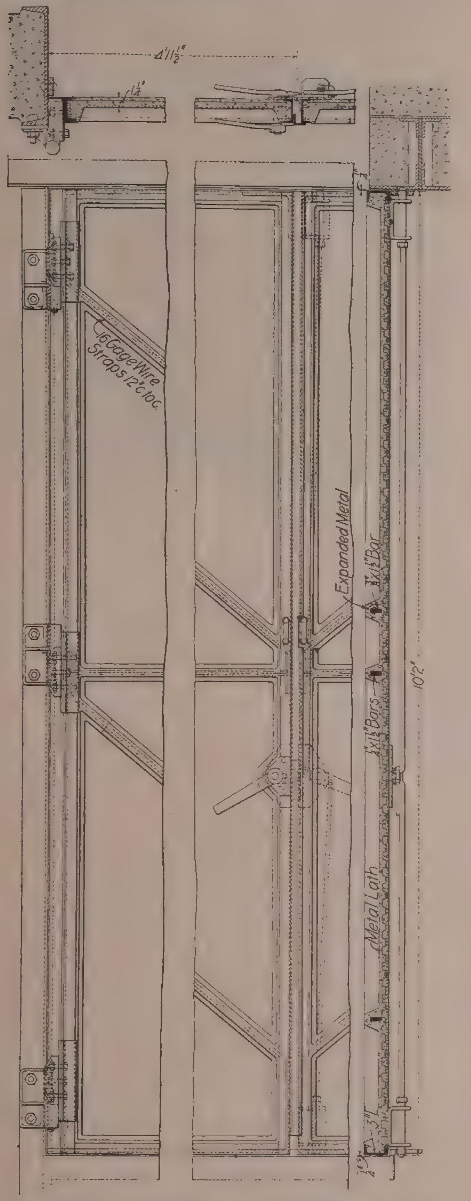
Two Mead-Morrison traveling bridges, permitting the 6-ton grab bucket to travel transversely 600 ft., serve the 1,000,000-ton storage pit where coal and limestone will be piled up for use during the six months' period when navigation is suspended. Legs of the bridge are supported on runways 382½ ft. apart. For the present the pit will be 1100 ft. long, but the runways will be extended ultimately to 1600 ft. Trains will enter on a trestle near one leg of the bridge. The trestle is supported on piers spaced 30 ft. on centers, 23½ ft. above the ground level. Much of the material on the site of this pit has been excavated for use in making concrete aggregate.

Water Supply

Water for the mills and furnaces is not repumped but wasted through the sewer, which discharges into a ravine to the west of the site. It is highly important, on account of the thin-shell furnaces, that there be no interruption of service. In consequence the supply of electrical energy to the 500-hp alternating-current motors driving the four 10,000,000-gal. De Laval two-stage 16-in. pumps may be supplied over three separate lines from different



DETAILS OF REINFORCED-CONCRETE CONDUIT FROM PUMPING STATION sources about the plant. In addition a 1,000,000-gal. unit is operated by direct current. The immediate consumption will probably be less than the output of three pumps, so there will always be one spare unit of large size out of commission. The plant and intake are designed for an ultimate extension to 100,000,000 gal. in twenty-four hours. Another safety measure is the elimination of priming difficulties by installing the pumps in a 32 x 107-ft. pit, 24 ft. deep, at such a level that the runners will be submerged at least 3 ft. Changes due to tide, wind and floods in the St. Louis River vary the water level by 5.5 ft.



DETAILS OF REINFORCED-CONCRETE DOOR

The walls and floor of the pump pit and forebay are designed and reinforced by hydrostatic upward pressure of 30 ft. on the floor slab. The basement floor is 5½ ft. thick, and the side walls are buttressed on both sides of the pit. Buttress walls, 2 ft. thick and 15 ft. center to center, form suction bays on the forebay side and extend out 19 ft. on the opposite side.

Horizontal bell-mouth suction pipes pierce the 2-ft. side wall of the 10 x 13-ft. suction chambers, with centers 4 ft. 3 in. above the floor of the latter. Entrance to these chambers is through two screen boxes covering two 3 x 3½-ft. openings in a 2-ft. vertical wall, 10 ft. back of inclined bar screens 1 in. apart. The screen boxes will be covered on three sides and the top with ⅛-in. mesh non-corrosive metal. The bars will be raked by hand within the building, a curtain wall extending below normal water level to form a seal directly under the building wall. The buttress partition walls stop at this point, but concrete I-beams extend to the back wall of the forebay and are reinforced to take either tension or compression stresses.

Discharge pipes from the horizontal-shaft pumps rise out of the pit over the wall and enter the top of the discharge mains through a special Y-connection. Two 30-in. National steel mains put together with Converse joints are laid to the plant in separate trenches about 800 ft. apart.

The pump-pit walls will be waterproofed on the inside, such leakage as occurs being drawn off temporarily through pipes inserted through drill holes.

INTAKE AND CONDUIT

The intake and conduit rest on treacherous material, particularly the intake, which was constructed within a row of United States steel sheet piling heavily braced. The intake is connected to the pumping station across the Northern Pacific Railroad right-of-way by a reinforced conduit 13½ ft. high by 13 ft. wide, of the modified parabolic section shown herewith.

The intake has an apron 2 ft. 9 in. thick, consisting of one-quarter of a circle of 28-ft. radius. No screen bars are provided, but an angle at the front of the apron was placed for use in case future operation should necessitate them. Sheet piling 15 ft. long was driven under the front of the mat to prevent undercutting.

Springs issuing from the bluff to the west of the plant are being developed to furnish drinking water. One concrete spring house has already been constructed. In it two 4-in. motor-driven plunger pumps discharge against air storage into the 4-in. mains, which eventually will be extended to all portions of the plant. This spring has a capacity of about 70,000 gal. per day.

Progress at the new plant has been such during the past season that it is likely operations can be initiated late next summer. Practically all buildings are erected, the blast furnaces are well along and much of the equipment is being installed.

The construction work is being carried on by the company forces, under the general supervision of George L. Reis, vice-president; K. C. Hoxie, chief engineer, and S. B. Sheldon, general superintendent.

THE LOWEST GRADE considered satisfactory for successful operation of flumes is approximately 1 per cent, but it is better to maintain a grade of from 2 to 5 per cent when possible, according to a statement by E. S. Bruce in Bulletin 87 of the U. S. Department of Agriculture. The maximum grade that can be used runs up to a very high pitch. Some flumes have been successfully operated for a short distance at a grade of 30 deg., but such a steep grade is undesirable as it is difficult to maintain a sufficient volume of water in the flume. The most satisfactory results in fluming are obtained at grades of from 2 to 10 per cent, and grades above 15 per cent should be avoided whenever possible.

How to Classify Rocks Quickly in the Field

Simple Tests and Compact Schedules of Characteristics Useful to the Engineer

By EDWIN C. ECKEL
Consulting Engineer, Washington, D. C.

IN the location or execution of engineering work it is frequently necessary to make some attempt to determine and name the rocks encountered in the course of the survey or construction. It is, in such cases, highly desirable that some method of determination, applicable in the field, should be available. Accurate determinations can, of course, be made later and confirmed by specialists if the matter is of sufficient interest, but in nine cases out of ten a rapid field test will suffice.

The compact schedules below have been prepared to serve this purpose. They do not require apparatus of any sort and the only reagent necessary, in some cases, is acid. When hydrochloric acid ("muriatic acid" of the drugstore) is available this can best be used. But in most cases it is well to remember, in default of such acid, that strong vinegar or lemon juice will be almost as good for testing purposes.

CRYSTALLINE

Crystalline rocks are made up of closely packed crystals of one or more minerals. In the first class are the following, in which only one mineral is visible:

Soft; scratched by finger nail; white to pinkish in color; no effervescence with acid. —*Gypsum.*

Harder, but easily scratched by knife; effervesces with acid.—*Limestone.*

Hard; scratched by knife with difficulty; green to black in color; probably a very basic igneous rock.—*Pyroxenite.*

In the second class are the following, in which two or more different minerals are visible.

Minerals arranged confusedly, with no trace of banding; light colored, commonly pinkish, reddish, bluish, light gray; predominant minerals, quartz, mica, feldspar, hornblende; probably one of the acid igneous rocks.—*Granite, Syenite.*

Dark colored; brown, dark gray, to almost black; quartz not noticeable; dense and heavy; probably one of the basic igneous rocks.—*Diorite, Gabbro, Trap.*

Minerals arranged in distinct bands; light colored; quartz, mica, feldspar visible.—*Granite Gneiss.*

Dark colored; green to black on fresh surface; often speckled on weathered outcrop. —*Hornblende Gneiss.*

GRANULAR

Granular rocks contain visible grains or pebbles, usually rounded, sometimes angular.

Soft, scratched easily by knife; colors white, buff, bluish; effervesces with acid; small rounded grains.—*Oölitic Limestone.*

Harder; not scratched easily if at all by knife; no effervescence with acid; grains siliceous; component grains small, rounded. —*Sandstone.*

Composed of rounded pebbles.—*Conglomerate.*

Composed of angular fragments.—*Breccia.*

SLATY

Slaty rocks break into thin flat slabs or slates.

Soft, scratched by finger nail; color, white to greenish or light green; soapy luster and feel.—*Chlorite Slate, Hydro-Mica Slate, Soapstone.*

Harder, but scratched easily by knife; clayey smell when breathed cn; colors gray, reddish, purple, black, greenish.—*Slate.*

Harder, not easily scratched by knife; showing crystalline structure on at least one fracture surface; mica flakes, quartz and hornblende usually noticeable.—*Metamorphic Schists.*

Light colored; principal minerals quartz and mica.—*Mica Schist.*

Dark colored; greenish to black when fresh; often speckled with white or yellow on weathered surface; no quartz visible.—*Hornblende Schist.*

COMPACT

In compact rocks the fractured surfaces show no trace of crystalline texture or of individual grains.

Soft; readily scratched by finger nail; effervesces readily with dilute acid; white to bluish or buff.—*Chalk.*

No effervescence with acid; white to pinkish.—*Gypsum.*

No effervescence; clayey smell when breathed on.—*Shale.*

Harder; but scratched easily by knife; effervesces with acid.—*Limestone.*

No effervescence; clayey smell when breathed on.—*Shale.*

Soapy feel; greenish color.—*Serpentine.*

Very hard; scratched by knife with difficulty; fracture dull; light or dark colors on fresh surface, but usually weathered white or yellow on outcrop; probably one of the denser fine-grained igneous rocks.—*Felsite* or *Basalt.*

Not scratched by knife; fracture smooth, often wavy; luster of fracture surface often waxy; in layers or beds interbedded with limestones, or in irregular nodular stringers in an individual limestone bed.—*Chert* or *Flint.*

Not scratched by knife; fracture curved; fracture surface glassy in luster, but usually slightly rough to touch; in beds or thick series of beds, associated commonly with schists or slates.—*Quartzite.*

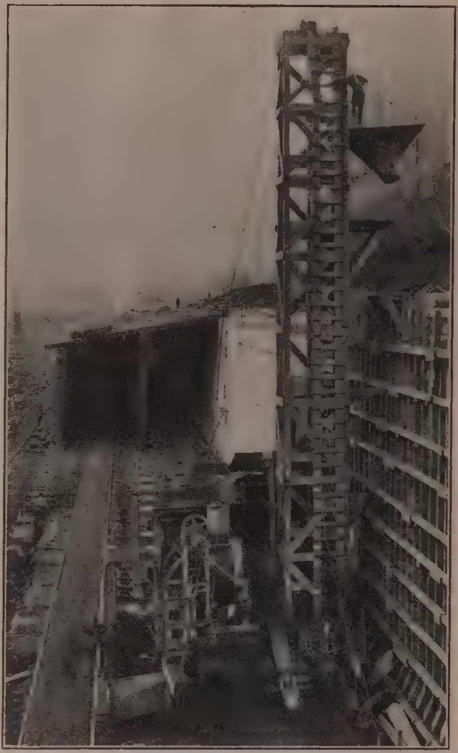
Combined Wood and Concrete Snowsheds in Cascades

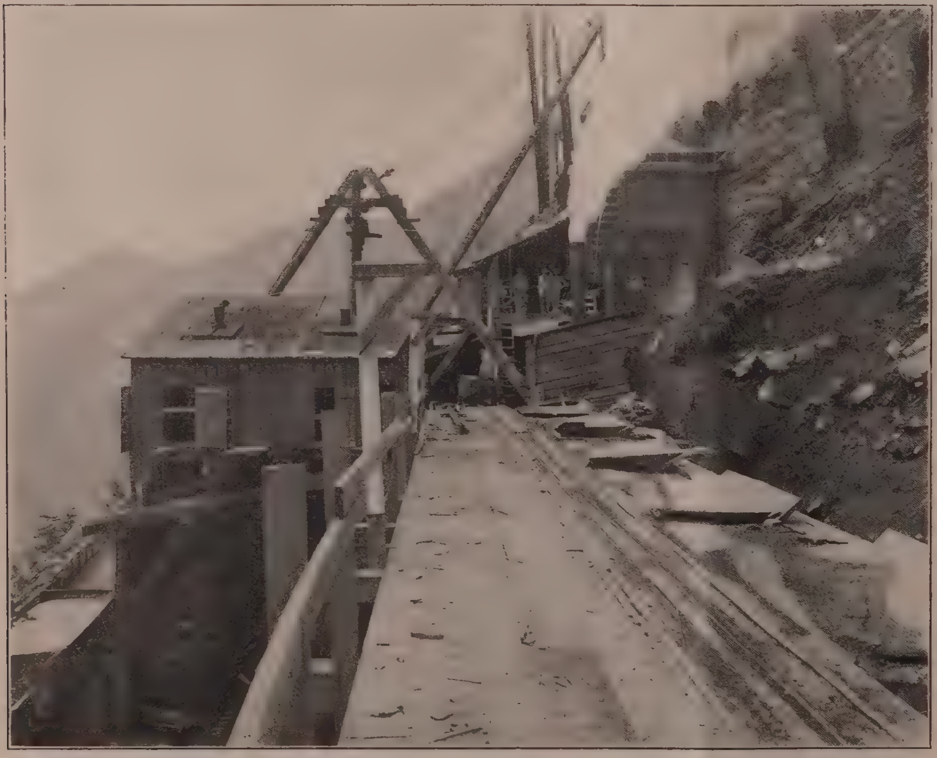
Construction Carried On under Difficulties Due to Small Clearances, Lack of Material and Scarcity of Labor

BUILDING of combined wood and concrete snowsheds in the mountain district of the Great Northern Railway was carried on during the season of 1913 under difficulties. Concrete aggregates had to be transported 100 mi., traffic had to be maintained while making small clearances for the concrete outfits, labor was hard to hold and stalled trains often stopped the work.

These snowsheds are located at three points on portions of the main line on the western side of the Cascades, 8 mi. apart by rail, but only 2½ mi. in an air line, the third camp being 900 ft. lower than the first one. Snows at the time of the Wellington slide are said to have fallen to a depth of more than 30 ft., and in 1912 slides occurred which were estimated to be from 30 to 75 ft. deep.

Erection of all-concrete sheds was considered not practicable on account of lack of available aggregate. Therefore, only the back retaining walls and the footings for two rows of wood posts, as shown in one of the photographs, were made of concrete. The roofs and front walls are of timber, of which 25,000,000 ft. b.m., were used. Construction details of the concrete work and timber erection for the two track sheds are shown in one of the photographs, which is a typical view of all three jobs. Each camp was equipped with portable concrete towers on which were a No. 12 Smith mixer, power chargers and steam hoisting engines. The boiler for the elevator hoisting engine supplied steam for the mixer engine. At one point a stationary plant was located on the sidehill above the shed and aggregate was conveyed to it by a derrick over the shed. Each camp also had two hoisting engines, derricks for wrecking the old single-track snowsheds and a locomotive with cars and steam shovel to excavate where there had been no shed. Footing forms were filled with concrete mixed in Smith hand machines. As the clearance was only 15 ft. between the wall and passing cars, the charging hopper could not be operated except when the tracks were clear.


USING PORTABLE CONCRETE TOWER


STATIONARY CONCRETE PLANT ON SIDEHILL, WITH AGGREGATE HANDLED OVER SNOWSHED

Involved in the construction were 56,000 cu. yd. of concrete, 1,000,000 ft. b.m. of form lumber, 100,000 cu. yd. of excavation, 75 per cent of which was rock, and about 150,000 cu. yd. of backfill.

In order to complete the work in the short summer season two shifts were employed, lighting of the work being accomplished by Milburn lamps. The first concrete was placed on June 25 and the last on Nov. 1. As wages were high and labor scarce, an extra-good camp with excellent board was necessary to retain the men, 200 of whom were going and coming each day during the height of the season.

The contractors for the snowsheds were Guthrie, McDougall & Company, of Portland, Ore., for whom J. C. Baxter was engineer in charge and H. J. King, resident superintendent. This work was under the general direction of the late Alexander Stewart, chief engineer of the Western lines of the Great Northern Railway.

"UNACCOUNTED FOR" WATER in Milwaukee was 11.43 per cent in 1913, 17.27 per cent in 1912 and 18.27 per cent in 1911. Proof that the unmetered and automatic fire-sprinkling systems caused most of the excessive increase is shown in his annual report by H. P. Bohmann, superintendent, since all services have been metered and the sprinkling systems sealed. The average daily increase for the years 1909 to 1912 were from 2,300,000 to 6,400,000 gal., while in 1913 the increase was only 370,000 gal. per day. An inspection of the 257 fire cisterns disclosed 53 that leaked, one of which wasted 40,000 gal. daily. Little evidence of electrolysis of water mains was found.

Cedars Rapids Hydroelectric Development

Construction by Unit Method of Reinforced Concrete of Power House Superstructure and Transformer House for Low-Head Power Plant

THE "unit method" of reinforced-concrete construction is used in building the superstructures of the power and transformer houses of the Cedars Rapids Manufacturing & Power Company's plant near Cedars, Que. The design of this development was described in an article in the Engineering Record of Oct. 25, 1913, page 461, and the construction work, up to the generator floor level, was the subject of another article appearing July 18, 1914, page 62. This article will deal with the construction method of the building itself and the transformer house.

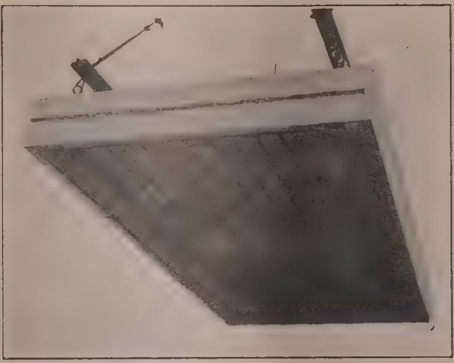
The power-house superstructure, which is 642 ft. 4 in. long, consists of thirty-five bays 16 ft. 8 in. and three bays 18 ft. 4 in. wide. The former spacing was selected because the generating units are spaced 50 ft. on centers, making three bays of the building per bay of substructure. Ultimately this building will be 1184 ft. long and have sixty-seven bays. The top width of the substructure is 132 ft., of which 3 ft. 6½ in. on the downstream side are a cantilevered reinforced-concrete walk. The superstructure has a depth of 124 ft. 11½ in. face to face of pilasters, leaving a 3½-ft. walk on the upstream side. The design and general arrangement of superstructure are shown in the accompanying drawings.

FRAMEWORK AND WALLS

It will be noted that the framework of the building is made of structural steel. As it was necessary to erect the columns and crane girders during winter so as to make the cranes available for the erection of the heavy machinery, the unit method of reinforced-concrete could not be used. The steel framework, therefore, was designed to hold the concrete units and ordered from the Phoenix Bridge & Iron Company, Montreal. The walls of the power house are made of double slabs 4 in. thick each. A 2-in. air space is left between the slabs so as to prevent condensation of moisture on the inside of the walls. The proper spacing is maintained by 2-in. projections along the sides and the lower edges of the inner slabs. The outside slabs have a 1-in. depression in the center so that the building has horizontal projections 9 in. wide, spaced 5 ft. 3 in. on centers. This arrangement was made so as to break up the monotony of the big panels.

Every panel is provided with a window on either the downstream or upstream side. The upstream windows are 39.56 ft. high and 15½ ft. wide, and are divided by a concrete beam 3.83 ft. high into upper semicircular parts of 7¾-ft. radius, and lower parts 27.98 ft. high.

On the downstream side, where the room for the auxiliary apparatus breaks up the wall, the windows are arranged in two parts, one on the generator-room wall above the projecting structure and the other on that structure. The upper window consists of a semicircular part of 7¾-ft. radius. The lower section is 9.31 ft. high, the two being separated by a transverse concrete panel 8.61 ft. wide. This separation of the windows was necessary because the auxiliary apparatus room has a third story at the exciter bay, necessitating the placing of the lower part of the window on the front of the wall.



METHOD OF HANDLING ROOF SLAB

The windows in the same panel of the apparatus room are both 7¾ ft. high and separated by a concrete beam 3.94 ft. high. All windows have United steel sashes made by the Trussed Concrete Steel Company, of Detroit.

Between bays there are pilasters 3 ft. wide, covering the flanges of the steel columns. These flanges are made of angle bars. Against the inside of the outstanding angle flanges bear the outer wall slabs, and as the columns project inside the building another set of angle bars, riveted to the web of the column, provide a bearing surface for the inner slabs. The outside slabs have loops made of ¼-in. round steel bars on both sides of the column flange. The pilaster blocks, which are 3 ft. high, have similar loops, so that when a number of them have been erected a ½-in. round bar is inserted through the loops, thus locking the pilaster blocks to the wall slabs. Later on the spaces between the concrete and the steel work are grouted.

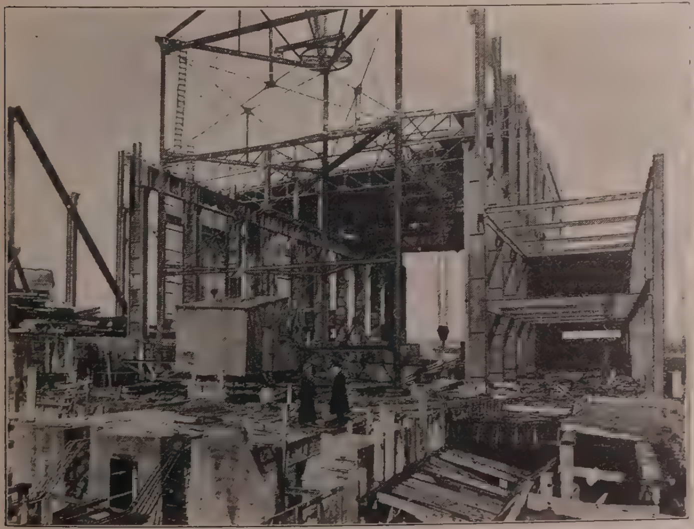
On the upstream side of the present building there will be two doors leading out on the walk. One is a 4 x 10-ft. hinged copper-plated door located in the permanent end panel of the building and the other is a large, double, copper-clad door, 12½ ft. wide and 14½ft. high, located in bay 24 from the permanent end.

Each end of the power house will have double, copper-clad doors, 15 ft. wide and 16 ft. high. The door in the north end accommodates a railroad track on which machinery and materials can be brought within reach of the cranes.

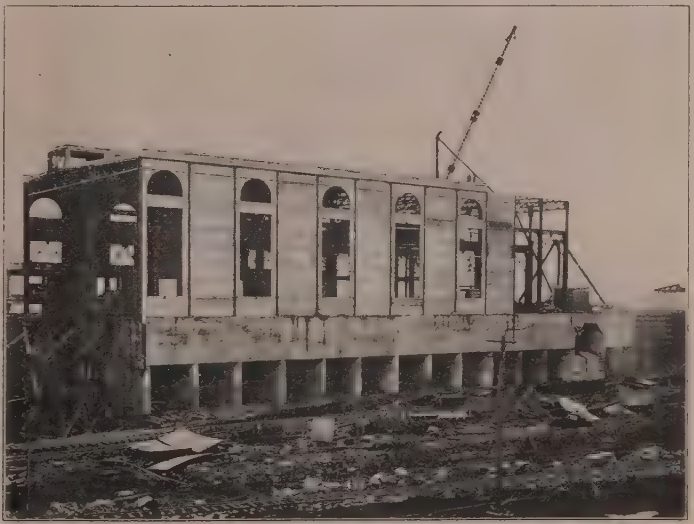
ROOF

As shown in the accompanying cross-section of the power house the 3-in. roof slabs are built integral with the supporting beams, half of each beam being molded with the adjacent panel. This arrangement gives these units a trough shape 16 in. thick over all. The units next to the coping of the roof, however, are 22 in. high on one side and 16 in. at the other, so that a gutter is formed along the roof. For the generator room the units are laid directly on the flanges of the upper chord of the roof trusses, while for the apparatus room, where no trusses are available, longitudinal 18-in. 42-lb. steel channels have been provided. These channels are riveted directly on the flanges of the columns.

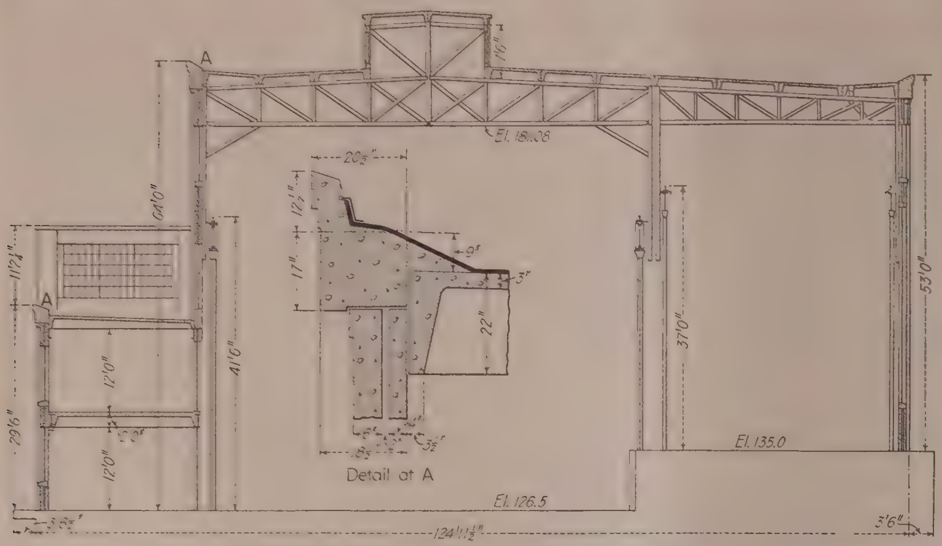
Two floors in the auxiliary apparatus structure are built monolithic. These are the only floors in the superstructure and one of them runs the whole length of the building while the other runs only three bays. The slabs are 3½ in. thick and are supported transversely and longitudinally on 20-in. 65-lb. I-beams. As the shear at the end connections of these beams is great, they are supported on riveted brackets secured to the web of the crane columns on one side and brackets riveted on to the face



SUPERSTRUCTURE AND TOWER DERRICK FOR ERECTION



UPSTREAM SIDE OF CEDARS RAPIDS POWER HOUSE



SECTION AND DETAIL SHOWING METHOD OF BUILDING POWER-HOUSE SUPERSTRUCTURE

of the outside columns on the other side. A 6-in. finish on top of the slabs makes the total depth of the floor 26 in. from bottom of beams to top of concrete.

Erection

All steel work and reinforced-concrete units are erected by a 10-ton derrick having an 85-ft. boom mounted on top of a triangular traveling tower. The construction of this tower is shown in one of the photographs. The track consists of three wooden stringers for the cast-steel rollers of the equalizers at the foot of the tower. As the derrick is located in one corner of the tower, the latter is guyed sideways so as to eliminate any danger of overturning.

To facilitate the handling of the units steel eyes are provided. There are two eyes in the upper ends of wall slabs and pilaster blocks so that these pieces hang vertically and can readily be placed. Floor and roof slabs, however, have four eyes each, so that they are lifted in a horizontal position. One of the illustrations shows the handling of a roof slab.

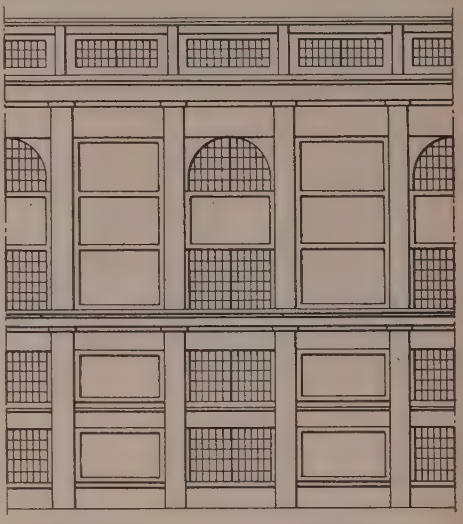
On the shore about 600 ft. from the power house is located the transformer house, in which the current will be stepped up from 6600 to 110,000 volts. It was found advantageous to build a separate transformer house because two separate organizations could then be occupied at the same time, and possible delays to the power house need not affect the progress of the building for the electrical equipment. The structure is 107 x 230 ft. in plan and its maximum height is 92½ ft.

The basement and the first floor are built of monolithic concrete spouted into wooden forms. From here up, however, the building is made of units, the general arrangement of which is shown in the accompanying cross-section. The most vital connections are those at the columns and the detail of design is, therefore, shown herewith.

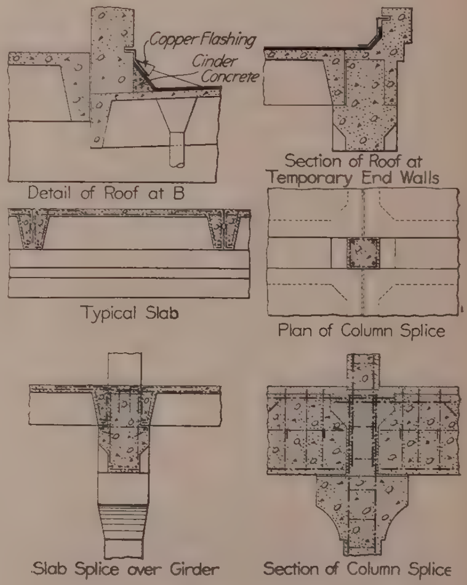
Manufacture of Units

In the article of July 18 was shown the location of the yard for casting and seasoning the units. At the upper end the units are cast in wooden forms, while the lower end is used for storage. The forms are made of wood and well oiled so as to prevent the concrete from adhering. The reinforcement, the major portion of which consists of corrugated and square twisted bars, is wired together, and the eyes used for lifting the units are carried well into the form and wired to the reinforcement. In the center of the yard is located a track for transporting the units. When a slab has set sufficiently it is picked up by a locomotive crane and carried to the storage.

Two mixers are used, one ¾-cu. yd. cube mixer and one ¾-cu. yd. Smith mixer. The concrete is made in the proportion of 1:2:4. When a batch is mixed it is dumped in a ¾-yd. Insley bucket, which by means of the locomotive crane deposits the concrete in the forms.



DOWNSTREAM VIEW OF SUPERSTRUCTURE

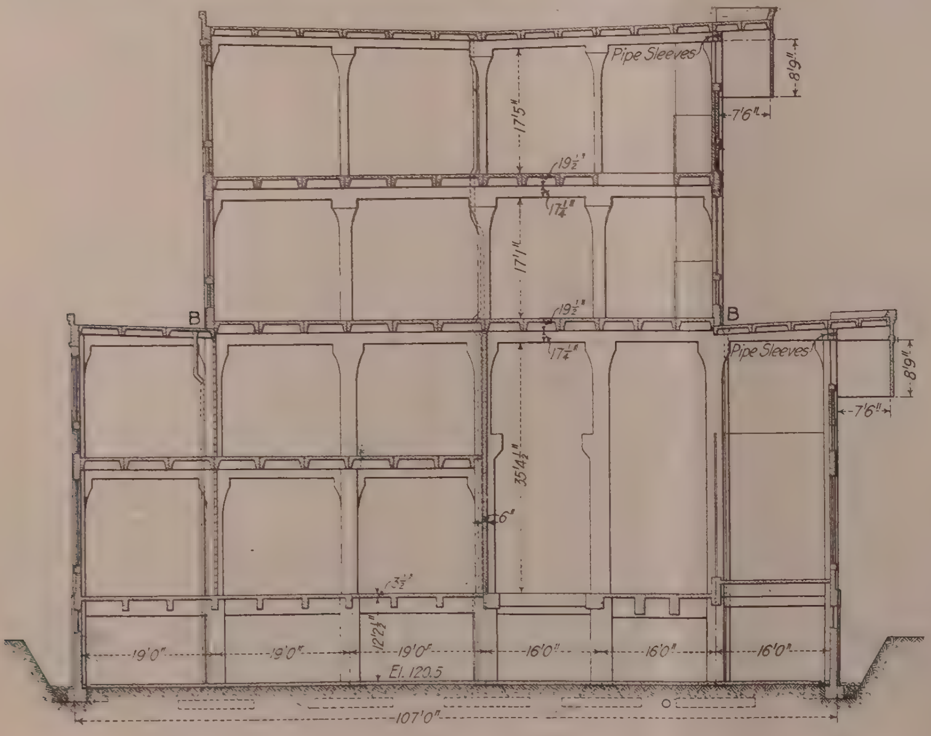


Detail of Roof at B
Section of Roof at Temporary End Walls
Typical Slab
Plan of Column Splice
Slab Splice over Girder
Section of Column Splice

DETAILS FOR TRANSFORMER HOUSE

Air spaces are required in the outer walls of the transformer house to prevent condensation on the inside of these walls. The walls are 10 in. thick, and as it was desirable to make them of single slabs instead of double, as done in the power house, these slabs are made hollow. The form for this space is made of sand, which later on, when the slab is lifted up, drops out.

In the storage yard the units are left to season until safe to handle before they are placed in either one of the buildings. The heavier units, like columns, girders and floor and roof slabs, are stacked up, one on top of the other, and separated by wooden lath, while the thinner wall slabs are left



SECTION OF TRANSFORMER HOUSE, SHOWING DETAILING FOR UNIT CONSTRUCTION standing on both sides of wooden bents, as shown in one of the photographs.

It is contemplated to have the buildings finished and the machinery installed so that electric energy may be delivered in December. The delivery of power on contracts already signed commences on Jan. 1, 1915.

The general design of these buildings was made by the engineering department of the Cedars Rapids Manufacturing & Power Company, of which Julian C. Smith is hydraulic engineer and R. M. Wilson, electrical engineer. The latter is assisted by Alexander Wilson. The resident engineer at the plant is George P. Hawley, with H. S. Slocum as assistant. The detailing of the concrete work of the buildings was done by the engineering department of the Unit Construction Company in St. Louis, of which J. E. Conzelman is chief engineer and vice-president and C. D. Morley general superintendent. J. E. Forwey is superintendent in charge of the construction and C. C. Smith is in charge of the Montreal office.

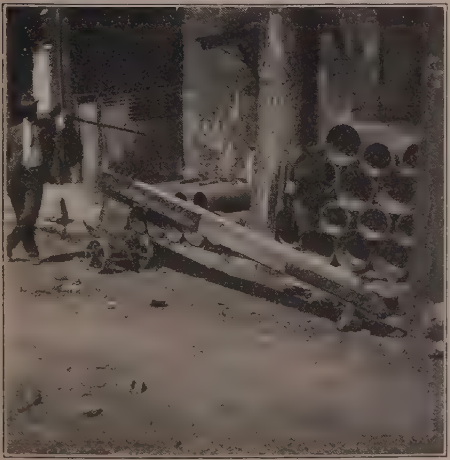
Using Plow to Open Pavements

Method Employed and Tabulation of Costs of Ripping Up Street Surfacing at Carlisle, Pa., to Permit Sewer Trenching

By C. A. BRYAN
Resident Engineer, Carlisle Sewerage Commission

PLOWING was the method adopted to break up stone-surfaced streets in Carlisle, Pa., as the preliminary operation in the construction of a new sewerage system. The beam of the plow, which was designed to loosen the material to a depth of 12 in., was made from a well-seasoned piece of hickory, 8 in. square and 12 ft. 4 in. long. The point or ripping device was rigidly attached to one end of the beam, and at the other a substantial iron eye was provided through which stout chains could be passed to attach the plow to the machine that was to haul it. The point or ripper was of manganese steel 1¼ in. thick and 37½ in. long and was beveled at each end; the bevel amounted to 6 in. at each end. The point projected 8 in. in front of the cutting edge and was rigidly attached to it by steel plates ½ in. thick placed on each side of the cutting edge and securely bolted to it by ¾-in. bolts. These same plates were similarly bolted to the point or ripper.

The cutting edge was made of a piece of steel 1¼ in. thick and 18½ in. square. The front or cutting edge was V-shaped in order to decrease the resistance of the plow in passing through the ground. As a piece it was rigidly attached to the under side of the beam by means of two 3 x 3½ x ⅜-in. angles. The short legs of both angles were bolted to the beam by ¾-in. bolts which passed entirely through the beam, and the cutting edge was in turn bolted to the long legs of these angles by ¾-in. bolts passing through both the 1¼-in. plate and the leg of the angles. The point or ripper was thus secured about 24 in. below the beam.


PLOW USED FOR OPENING PAVEMENT

To strengthen the sides of the beam a piece of band iron 6 x 60 x ½ in. was securely bolted to each side at the rear of the plow, and two narrower pieces served the same purpose at the front end. The plow was guided by means of handles.

To facilitate moving the plow from place to place a two-wheeled truck was built. This truck carried a sort of iron loop into which the point or ripper could be slipped, thus raising it about 6 in. above the ground and allowing the plow to be easily hauled about. An iron wheel about 6 in. in diameter was attached near the front end of the plow.

The plow was designed to be hauled behind a steam roller, to which it was attached by means of heavy iron chains, as shown in one of the illustrations. The method of operation was as follows: The center line of the sewer trench was first located by the engineer, who drove nails 50 ft. apart in the surface of the street. A line was then stretched from nail to nail and a laborer passed along it, distributing red clay as he moved. This line of clay clearly defined the center line of the ditch and acted as a guide for both the man operating the roller and the men steering the plow. A hole was then dug at one end of the block to be ripped up and made large enough to set the plow into it. The plow was put into place, the engine attached and the plow dragged slowly along, following the center line as marked.

WORKING PLOW WITH STEAM ROLLER

Generally it was found necessary to drag three or four times over the ditch in order to loosen completely the stone surfacing. In this way it was found that the stone surfacing would be loosened for a width of about 1 ft. each side of the center line of the ditch and for a depth of between 1 and 1½ ft. below the surface of the street. The gang was composed of four or five men—one man to operate the roller, two to steer the plow and one or two on the front end of the plow in order to hold it down, as it always exhibited a tendency to rise. Usually between two and three hours were required to plow up the surfacing on a block averaging 500 ft. in length.

RESULT OBTAINED WITH THE PLOW

The results obtained were very successful, as the plow left the surfacing materials so thoroughly loosened that they could be easily removed with pick and shovel. Stones fully 1 cu. ft. in size were removed by the plow. It was found that the plow would loosen almost any stone that was used in constructing a road, but it could not, of course, affect solid rock. It would also rip up without much difficulty crosswalks built of grouted paving brick. On some streets considerable difficulty was experienced, this being especially noticeable on those streets where the solid rock projected into the street surface. In one instance this resulted in the breaking of the plow beam. It was also found that the cutting edges required frequent sharpening in order to get the best results.

The accompanying table gives the estimated cost to the contractor of the work of ripping up the street surfaces on all the blocks on which this plow was used.

Costs of Ripping Up Pavements with Plow

Foreman, 147 hr. at 25 cents	$36.75
Roller, including engineer, oil, fuel, etc., 210 hr. at 50 cents	105.00
Labor, 450 hr. at 17½ cents	78.75
Original cost of plow	55.00
Repairs (estimated)	20.00
Total cost to contractor	$295.50
Linear feet of street surfacing ripped up (estimated)	22,000
Estimated cost per linear foot of trench	$0.015

The contract for the construction of the sewerage system was awarded to the H. C. Brooks Company, of Martinsburg, W. Va. H. M. Sherwood was in charge of the construction work. The plow was built by the Carlisle Axle Works under the direction of Mr. Brooks. The writer was resident engineer in charge of the work.

BLEACH applied to Milwaukee water varied in 1913 from 3 to 8½ lb. per 1,000,000 gal., according to the annual report of the water department. A permanent plant of concrete tanks and measuring equipment housed in a brick structure was put into operation Sept. 29, 1913. The typhoid death rate of 11 per 100,000 population is the lowest in thirty-nine years. While hypochlorite has been used since June, 1910, its application was intermittent until 1913. Typhoid death rates beginning with 1903 to date are 17, 13, 20, 29, 25, 15, 21, 45, 19, 25 and 11.

Tunnel Damage at Poe Lock

Water Entering Behind Unanchored Timber Lining of Conduit Tears It Out, Necessitating Reconstruction

THE inner layer of the timber lining of the north filling conduit of the Poe Lock, St. Marys Falls Canal, was found to be damaged when the annual inspection was made after the close of the navigation season of 1913. A description of the damage and the means proposed for the repairs are given in the "Professional Memoirs" of the Corps of Engineers, U. S. A., for July-August, by Lieut.-Col. Mason M. Patrick.

There are six filling tunnels, each 8 x 8 ft., passing through the miter wall of the upper lock gate and under the floor of the lock with openings through the floor. The two outside tunnels extend to about the middle of the lock. The upper end of each tunnel is closed by a butterfly valve. The sides of the tunnels are 2 ft. thick, made by bolting 12-in. square timbers one on top of the other. The lock floor over tunnels is held down by long 1½-in. fox wedged bolts extending from the transverse timbers in the floor through the tunnels about 6 in. from the side walls and about 6 ft. into the rock. These bolts are subject to considerable vibration, which has loosened the nuts and in many cases there are no vestiges of the threads on the portions of the bolts which the nuts occupied. Such bolts are repaired by cutting off the top halves, replacing them by half lengths having heads on their upper ends and connected with the lower halves by turnbuckles.

TUNNEL LINING

The tunnel lining is flush with the masonry of the piers of the miter wall. The timbers in each layer of this lining are drift-bolted together, but the two layers are not connected nor were the linings of the outer tunnels anchored to the masonry behind them. When the north tunnel was entered at its lower end for the usual inspection, there was found a confused mass of 12 x 12-in. timbers, evidently at one time part of the tunnel lining. The inner layer of this tunnel lining for about 20 ft. from its upper end had been torn from its place and much of it had been swept down to the lower end. A few timbers had been displaced but not entirely detached. The fox wedged bolts had been broken and bent. For a further distance of 100 ft. this outer layer of the lining had been bowed out sufficiently to bend these long bolts. The inner layer of the lining seems to have been undisturbed. Where the outer layer has been torn away there is no evidence that the inner layer has been moved at all.

Evidently the inrushing water found its way between the two layers and separated them, tearing out the outer layer for a portion of its length.

It is proposed to repair the damage by replacing the timbers, placing along their inner faces at proper intervals vertical channels held in place by fox wedged bolts extending through both layers of the lining and into the masonry behind them. Where these timbers are bowed out, they will be jacked back into place and secured by channels in the same manner. This method is cheaper than the substitution of concrete for the timber tunnel lining

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Ultimate Strength of Carbon-Steel Models of Quebec Bridge Members

Records and Diagrams of Destructive Tests; Anchor Arm Lower Chord Models Buckled under Unit Load of 50,886 Pounds

AMONG the eighteen tension and compression tests of models to a scale about one-quarter the natural size of the members of the trusses in the 1800-ft. span of the new Quebec Bridge which have recently been made there are four sets on carbon-steel sections of the compression lower chords L10-12 in the anchor arm, located as shown by the diagram published in the Engineering Record of March 21, page 333. These members have the same position in the truss as those which failed in the Quebec Bridge collapse several years ago. In the new bridge structure they are proportioned for heavier total loads and for smaller unit stresses, and their design differs materially from that of the corresponding members in the former structure, which had four deep vertical webs connected only by end diaphragms and by top and bottom chord latticing.

The models of the new bridge members have been made and tested in the interest of the accepted design which has been prepared after investigation of similar members in the other largest recent spans and may be considered as the latest development in compression members to resist excessively heavy stresses in long-span trusses.

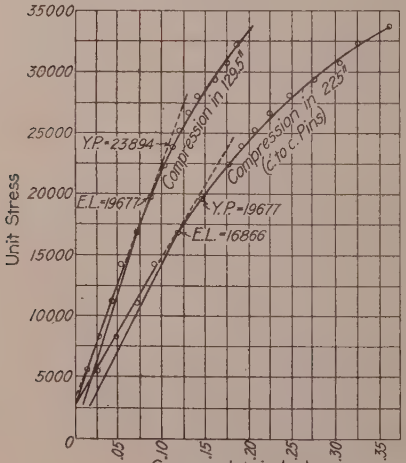
MODELS

The models, about 30 in. wide, 18¾ in. deep and 18 ft. 9 in. long, weighed from 7610 to 7650 lb. each and were marked TX13-A, B, 1 and 2. They had four deep vertical webs connected by full-length horizontal longitudinal latticing and tie plates on the center line and on the top and bottom flanges. They had a ratio of $L/R = 38$ and a nominal cross-sectional area of 70.65 sq. in., which, however, owing to the inaccuracy of rolling the plates and angles, was reduced to an actual area of about 69 sq. in., making the sections a little more than 2 per cent light. Specimen tests made from the plates and angles showed an elastic limit varying from 39,780 to 42,700 lb. They had half-hole bearings in both ends of the webs, which were reinforced to give 52 sq. in. of bearing for the 6½-in. pins.

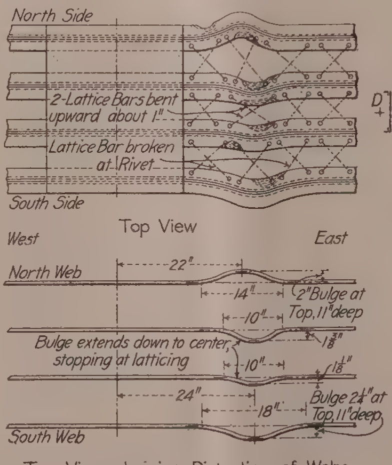
The accompanying illustrations are of special interest in that they include the first published details of the bottom chords of this great span. All of the models were symmetrical about their center transverse axes and those marked A were made with both ends exactly like the left-hand end of the illustration, with the flanges of the center webs connected by lattice bars. The models marked B were exact duplicates, except that they were made with both ends like the right-hand end of the illustration, with the flanges of the middle webs connected by tie plates. The models were supported by end pins alone, without intermediate counterweights to carry part of the weight.

GENERAL RESULTS

The elastic limit of the models, taken as a whole, varied from 16,866 lb. to 22,441 lb., and in all except the last case was much lower than the elastic limit determined on a long length measured in the center of the member. One model failed with a maximum unit load of 39,359 lb. The other

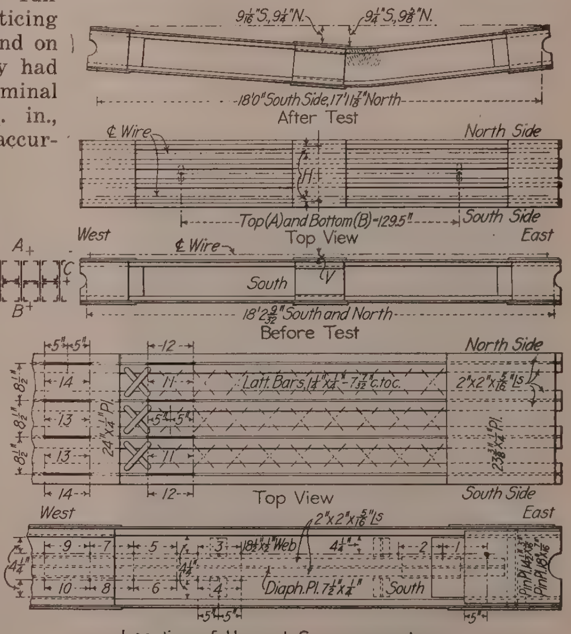


Compression in Inches



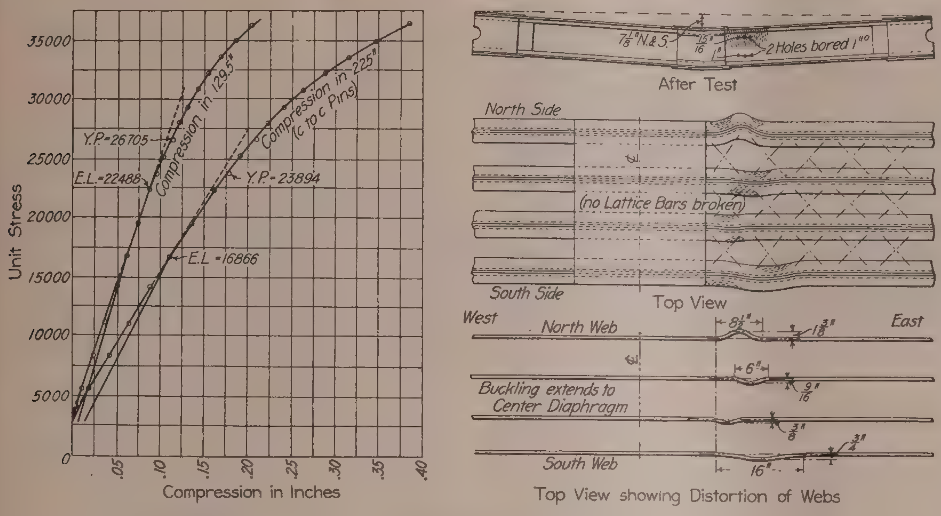
North Side
2-Lattice Bars bent upward about 1"
Lattice Bar broken at Rivet
South Side
Top View
West
East
North Web
22"
14"
2" Bulge at Top, 11" deep
10"
1 3/8"
Bulge extends down to center, stopping at latticing
10"
1 1/8"
24"
18"
Bulge 2¼" at Top, 11" deep
South Web

Top View showing Distortion of Webs

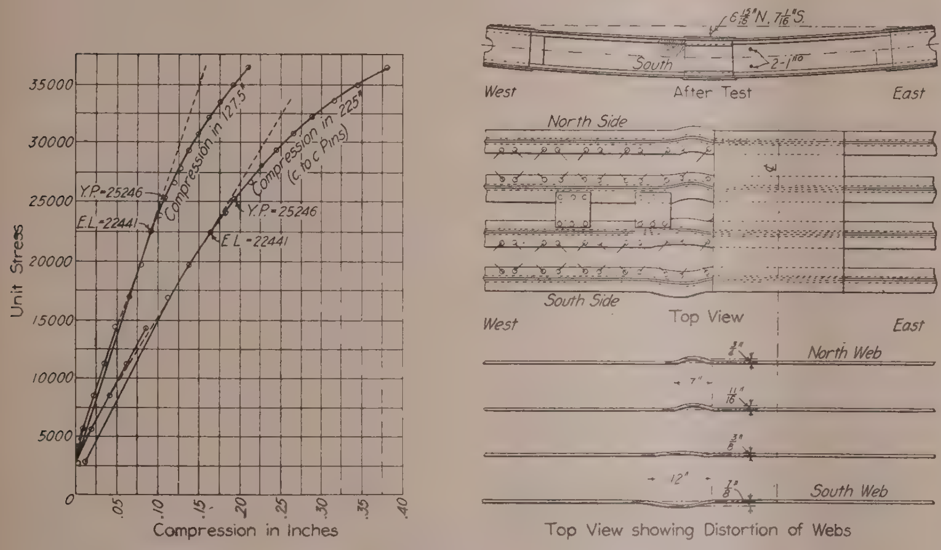


Location of Howard Compressometers

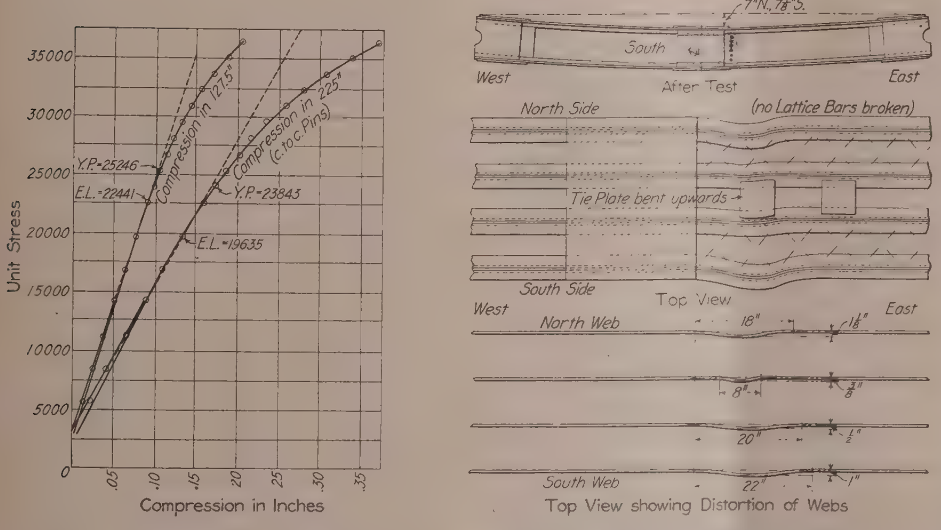
COMPRESSION DIAGRAM AND MANNER OF FAILURE OF MODEL TX13A-1 AND LOCATION OF COMPRESSOMETERS



COMPRESSION DIAGRAM AND MANNER OF FAILURE OF MODEL TX13A-2



COMPRESSION DIAGRAM AND MANNER OF FAILURE OF MODEL TX13B-1



COMPRESSION DIAGRAM AND MANNER OF FAILURE OF MODEL TX13B-2

COMPRESSION TESTS OF CARBON-STEEL LOWER-CHORD MODELS

Mark	ELASTIC LIMIT In portion of member at center	In full length	Ultimate strength	Percentage model is light of nominal weight	MAX. VERT. DEFLECTION AT CENTER, INCHES Before test	After test	Date of test, 1913	TIME OF TEST READINGS First	Last
	Pounds per square inch								
TX13A1	19,677	16,866	39,359	2.51	3/64	7 4/32	February 28	1.55 P.M.	5.26 P.M.
TX13A2.	22,488	16,866	43,164	2.51	1/32	7 1/8	March 4	9.27 A.M.	1.14 P.M.
TX13B1.	22,441	22,441	43,060	2.29	21/64	. . . . . .	March 3	9.05 A.M.	1.05 P.M.
TX13B2.	22,441	19,635	50,886	2.3	1/32	7 1/8	March 5	1.10 P.M.	4.45 P.M.

three models endured without failure several repetitions of the maximum load of the 2,800,000-lb. capacity machine. Afterward they were weakened by holes drilled through the webs near the center of the piece and failed under loads of from 43,060 lb. to 50,886 lb. per square inch computed on the reduced sections.

FORMER SIMILAR TESTS

A very interesting comparison can be drawn between the results of these tests and those published in the Engineering Record of Nov. 19, 1910, page 564, on nickel-steel models of the official design of the same bridge, which has since been superseded by the somewhat modified contract design now being fabricated. The nickel-steel models were about twice as long as these and had a considerably smaller cross-sectional area. They developed a maximum elastic limit of 43,947 lb. in the full-length measurements and had an ultimate strength beyond the capacity of the testing machine, so that it was necessary to weaken them also by boring holes, which reduced the cross-sectional area so that failure was finally produced by a maximum ultimate load of 63,990 lb. computed on the reduced section.

MANNER OF FAILURE

Model TX13A-1 failed by lateral deflection in the planes of the webs and by buckling of the webs near the center of the piece. Diagram of locations of the strain gages and of the compressometers is here given. The locations of the compressometers were the same for all of the tests in this set, except that for the models marked B the distance measured on the latticed faces was only 127.5 in.

Model TX13A-2 was compressed to the full capacity of the machine, which produced a stress of 40,657 lb. without failure. Afterward two 1-in. holes were bored through each of the four webs near the center of the piece, reducing the section from 68.87 to 64.87 sq. in., and the same maximum total load was applied three times at high speed, producing a unit stress of 43,164 lb. per sq. in., and on the third application caused the failure of the model by a downward deflection and the buckling of the webs near the center. The strain-gage locations were all made on one latticed face of the model, none of them being made on the webs.

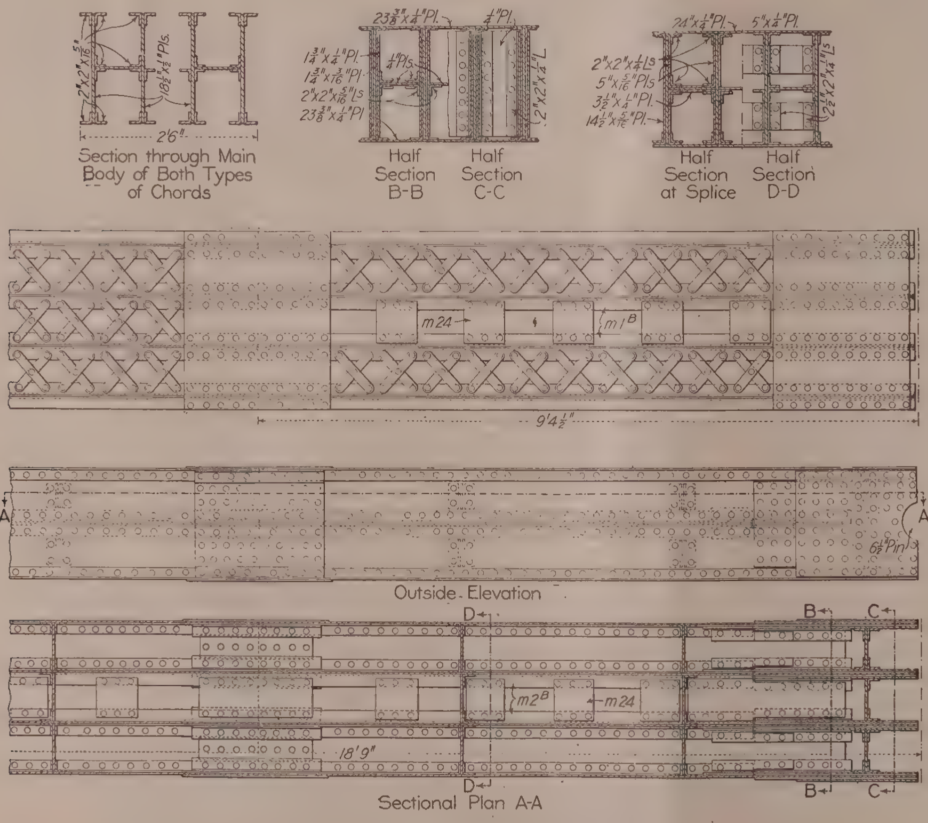
Model TX13B-1 endured without failure the full load of the machine, producing a compression of 40,565 lb., after which two 1-in. holes were bored through all four of the webs near the center of the model, reducing the sectional area from 69.026 to 65.026 sq. in., and the same maximum load was applied again seven times at high speed, producing a stress of 43,060 lb. and finally causing the failure of the piece by bending at the center in the plane of the webs and by the buckling of the webs near the center. The strain gages were applied at ten points on the webs on both sides of the model and at eight points at the upper latticed face.

Model TX13B-2 was loaded to the full capacity of the machine, gage 867.7, producing an ultimate compression of 40,565 lb. on the full section of the piece, which it endured without failure. The pressure was released and two 1-in. holes were bored through all four webs, reducing the sectional area 4 in., and the same loading was repeated ten times. As this did not produce failure, two more holes were bored

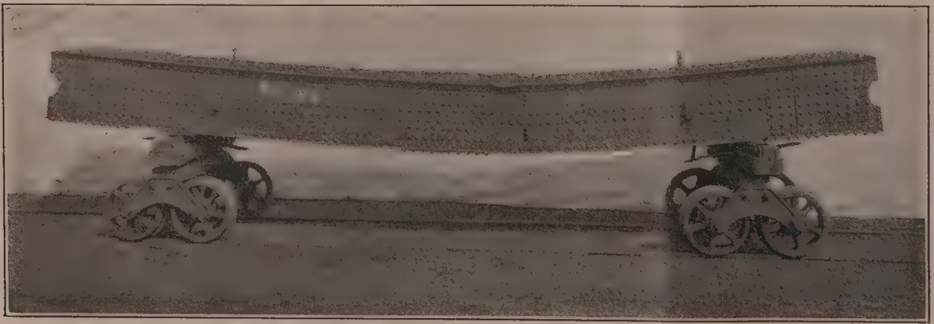
MODEL TX13A-1 FAILED UNDER TEST LOAD OF 39,359 POUNDS PER SQUARE INCH

and the same loading repeated several times. Finally the section was reduced 14 in. by boring six holes through the two south webs and eight holes through the two north webs and the same load again slowly applied and maintained for 10 min., producing a compression of 50,886 lb. per square inch on its reduced cross-section of 55.026 sq. in. After 10 min. application this load caused the failure of the model by bending transversely at the center in the plane of the webs, which were buckled somewhat at the location of the holes.

C. N. Monsarrat is chairman and chief engineer and Ralph Modjeski and C. C. Schneider are the members of the board of engineers of the Quebec Bridge. The tests were made for the St. Lawrence Bridge Company, of Montreal, contractor for the superstructure of the new Quebec Bridge, by the Phoenix Bridge Company, of which John Sterling Deans is chief engineer, P. L. Szlapka, designing engineer, and Charles Scheidl, assistant engineer in charge. The tests were made under the direction and personal supervision of James Howard, engineer-physicist, Bureau of Standards, Washington, D. C.



DETAIL OF CARBON-STEEL MODEL OF ANCHOR ARM LOWER-CHORD SECTION



MODEL TX13B-2 FAILED UNDER TEST LOAD OF 50,886 POUNDS PER SQUARE INCH

Pittsburgh Flood Commission Answers Sir William Willcocks

Calls Eminent English Engineer to Task on Ethical Grounds for Expressing Opinions without Fully Informing Himself as to Facts

IN the "Proceedings" of the Engineers' Society of Western Pennsylvania, Vol. 30, No. 2, March 14, 1914, the paper read by Sir William Willcocks on "How the Ancients Would Have Controlled the Mississippi and Its Tributaries" (see Engineering Record, June 27, page 723, and July 4, page 16), contains certain references to the report of the Flood Commission of Pittsburgh. To the criticism of Sir William the members of the engineering committee of the commission—George I. Davison, Paul Didier, George M. Lehman, Julian Kennedy, Morris Knowles, E. K. Morse, Emil Swensson and W. G. Williams—have made the following reply:

"The privilege of an engineer to discuss the work and opinions of his professional brethren is well known and this may even become a duty, if differing profoundly upon important public policies affecting the safety, health and general welfare of the people. There is, however, a duty, fully as grave and customarily imposed, and that is that one shall completely inform himself before criticising the position and opinions of others. This is particularly true and the duty is all the more important when the critic occupies a world-wide position of renown and international standing in the profession.

FLOOD-COMMISSION REPORT

"As, in his address delivered at the invitation of the Engineers' Society of Western Pennsylvania, certain statements appear to attack the methods and findings of the engineering work of the Pittsburgh Flood Commission, the members of the engineering committee believe it a duty to the profession, as well as to their associates on the commission, to answer the sensational criticisms contained therein. The remarks of the author evidently were not based upon an accurate reading of the flood commission report. Had he carefully read and considered such report or obtained correct information in relation thereto, it is believed that his views upon the matter of flood control, as applied to our local rivers, would have been materially modified.

"The author undertakes to impress his readers that his opinions were based upon personal observations and at the same time to lodge a charge of incomplete investigation against those who have been responsible for the flood commission report. For instance, he says: 'I never saw such bad looking stuff for reservoirs and I found that no one had bored down 1 ft. to see.'

"As a matter of fact the author never visited a single reservoir site and fails to show the source of information, hearsay or otherwise, upon which he was relying for his extravagant statements. Upon the other hand, members of the engineering committee of the Pittsburgh Flood Commission visited the site of each proposed dam and satisfied themselves fully as to the presence of natural rock foundation.

"Again the author says: 'And your condition is such that if you had a high flood and a reservoir with 150 ft. head on the hills above you and it happened to breach and come on top of the flood all of the disasters you know of to-day would be but child's play. And if you had two in the same valley and the upper one burst and came down on the lower one, and the two came together, you might open the early chapters of Genesis and begin reading about Noah's flood to comfort you.' The purpose of the author's pleading is not clear. If he is adverse to dams of 150 ft. or more in height because these are dangerous, we respectfully call attention to the fact that the highest dam proposed by the commission is 143 ft. in height and the average of the seventeen dams proposed is 93 ft. However, we further note that there are several dams in this country of more than 200 ft. in height, and the National Government is now engaged in the construction of a dam of 412 ft. in height on the Sun River in Montana. Height simply requires adequate design and strength.

"In the paper the following statement occurs: 'This horizontal sandstone you have here and shale in alternating strata is considered the worst foundation for reservoirs of anything in the world. More accidents to big reservoirs have happened on it than any other. If the strata are inclined at a steep angle and you build your dam on it, it rests on the hard particles and the weak strata are more or less ignored.' As the author very shrewdly abstains from giving definite information as to who holds these absurd opinions as to our local geological conditions, and as we are addressing those who have knowledge of these matters, comment is unnecessary.

Borings

"The author states: 'In your book you show that you spent money on many things, but not one penny to show what your foundations are, and all the rest of it is worth nothing until you are sure of your foundation. You ought to spend some $10,000 for drills and take two of the nearest sites and expose the foundation and see if you can build a dam there. If you find real good foundation your difficulties will be at an end.' Engineers of ability and experience engaged upon a preliminary study and general design judge by the surrounding outcropping rock, which indicates geological conditions. Thus the feasibility of location is determined and customary allowance made in estimates of costs for depths of foundations. Later drillings will determine actual design and depth. Without undertaking to give detail answer to this criticism and show how the money was spent, all of which is stated in the report thus mentioned, the engineers of the flood commission, after personal inspection of the sites, considered that it was unnecessary to spend any money for borings for the purpose of their preliminary report. But it goes without saying that before actual sites would be determined and building begun such borings would, of course, be made by those responsible for the design and construction.

Hydraulic Measurements

"It is further stated: 'In a recent book which you have written on this reservoir question I see that the quantity of water you consider necessary to impound in these reservoirs seems to be in excess of what you need. In all the calculations it has been assumed that when the river rises its discharge increases up to its maximum stage. As in all these estimates you have allowed for an increasing discharge and not reduced by half for the falling gage, a much smaller quantity of water than you have assumed would, I think, suffice to shelter you from these hours of high flood which produce all the worry.' This appears to be a criticism of the use, by the flood commission engineers in their report, of a greater factor of safety than the author thinks necessary. It may be stated that the hydraulic measurements upon which the quantities were determined were based upon the careful methods of the U. S. Geological Survey and of the Pennsylvania Water Supply Commission and many of the stations were operated in co-operation. Full and complete allowances were made in the rating curves, notwithstanding the above quoted comment, for the fact that river velocities are not the same upon rising and falling stages. This statement can be verified by reference to the rating diagrams in the report.

"The weakness of the author's paper and of the apparent criticisms of the flood commission's report is due to two facts: The opinions expressed in the report were based upon five years' study by engineers who possessed full knowledge of local circumstances and who therefore do know, whereas the remarks of the author emanated from a necessarily cursory and hasty examination of the subject and conditions.

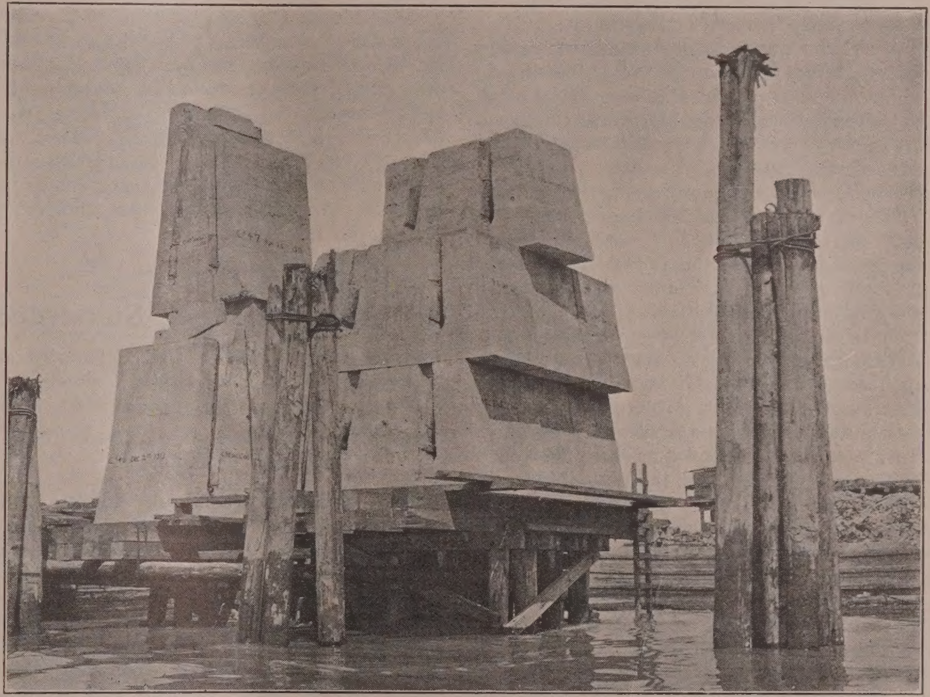
"The engineers of the Pittsburgh Flood Commission have the highest appreciation of the professional and ethical responsibility of opinions upon such a question as flood control and regret that so eminent an engineer should undertake to speak to the world upon such an important subject without fully informing himself as to the data from which he draws his conclusions."

ABOUT 65 per cent of the Federal railways of Switzerland rest on steel ties and 35 per cent on wooden ties, according to a recent consular report. The steel ties are in the form of inverted troughs with the ends bent down to close the troughs. The tie complete weighs about 160 lb.

Bearing of Piles in Silt

Long-Continued Loading Tests of Groups of 90-Foot Lagged Wooden Piles for Municipal Piers in New York City

THE floor of the municipal pier at Forty-sixth Street and North River, New York City, will carry a steel superstructure and be subjected to heavy loading. Part of it will be supported on solid fill, but the outer end will be carried on wooden piles driven in deep water and very soft silt which affords little or no sustaining power, allowing the piles to sink into it for a long distance without driving. Tests of ordinary wooden piles showed practically no bearing power, while tests of similar piles, lagged, indicated that the required bearing capacity could be obtained. Ordinarily these indications, combined with information derived from tests of piles driven under similar conditions for other piers, would have sufficed, but in this case, on account of the very heavy loads to be provided for and space and opportunity conveniently available, it was decided to make full-size tests with exact loading required to support the floor and the two-story steel shed.



WORKING LOAD TEST OF LAGGED PILES IN SOFT DEEP MUD

Tests

Groups of piles connected and covered with a platform to form eventually part of the permanent pier floor were driven last winter and loaded with the full weight which they were destined to support. The first test was made on a group of sixteen wooden piles 90 ft. long, every other one of which was reinforced by four 5 x 6-in. lagging pieces 30 ft. long bolted to it at the foot. These were driven in about 20 ft. of water and were loaded by concrete wall blocks having a total weight of about 240 tons, which they did not satisfactorily support, showing too much settlement for the steelwork.

A second test was made with another group of sixteen piles driven adjacent to the first one, but having every pile lagged and having the piles distributed over a larger area. These piles were also braced and capped and loaded with concrete blocks, which remained several months in position without causing too much settlement. The second test afforded sufficient data for the design of the pier foundations. The tests were made by the Department of Docks, of which R. A. C. Smith is commissioner and C. W. Staniford, chief engineer.

Slag Paving Block in Washington and Baltimore

Product of English Iron Furnaces Successfully Used in Railway-Track Areas and on Heavy Grades

ABOUT 3000 tons of "Scoria" paving block, a slag product derived from the iron furnaces at Middlesborough, England, were recently brought to this country. Half of the cargo was unloaded at Baltimore and the remainder taken to Washington, in which cities a considerable use of the brick has already been made. R. Keith Compton, chairman of the paving commission of the city of Baltimore, states that the blocks are being used as liners along the railway tracks on Howard and Baltimore Streets in that city, as well as for footings around curves and switches and for several squares of track area on Baltimore Street where the grade is from 3 to 4½ per cent, the remainder of the street being sheet asphalt. Mr. Compton assigns the merit of the block on heavy grades to the fact that it is not as slippery as asphalt, although a pavement with a fairly smooth surface may be built. He states that the product has also been used very successfully in Washington in similar locations, but of course its short career in this country does not allow claims of superiority over other hard materials for track work.

The specifications used in Baltimore for the Scoria blocks are as follows:

"The blocks shall consist only of iron slag blocks from blast furnace slag.

"The blocks shall be of regular form and size and each of same dimension.

"The size of the blocks shall be 6 to 9 in. in length, 3½ in. wide and 4 in. deep.

"They shall be all hard, durable and uniform in quality and dimensions.

"All blocks must have good, flat, even beds and upper surfaces, with vertical sides.

"Blocks must be sound and perfect in all respects, free from cracks, hollows and other defects.

"In appearance the blocks shall show an opaque stone-like surface, and below the outer skin a blue porcelainic color.

"The bidder shall leave at the office of the chief engineer at or before the time for submitting bids a specimen block of the size and quality described in these specifications, labeled with the name of the bidder and place of manufacture.

"The chief engineer reserves the right to have the contractor use square-edge blocks as liners if they are obtainable.

"The blocks will be laid on a ½-in. mortar cushion consisting of sand and cement in the proportion of one part Portland cement and three parts clean, sharp sand."

The process of making the blocks was described in the Engineering Record of Aug. 30, 1913, page 241. The present shipment will be the fourth consignment to be brought to Washington and Baltimore within the last four years.

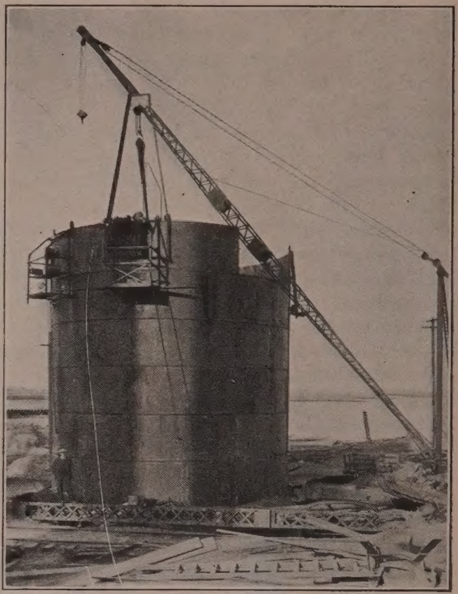
A 100-Foot Steel Standpipe

Structure at Long Beach, Cal., Is Founded on Piles and Was Erected without Scaffolding

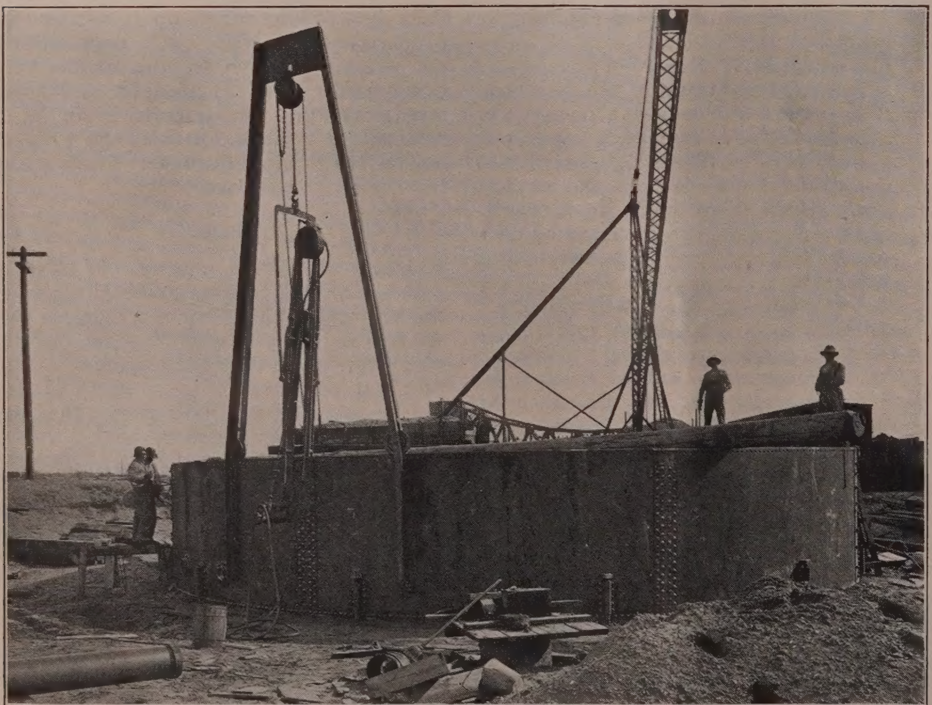
By EDWIN H. WARNER

Resident Engineer, Southern California Edison Company, Los Angeles

THE Southern California Edison Company has erected at Long Beach, Cal., a steel standpipe 36 ft. in diameter and 100 ft. high for use in connection with its 84,000 kva steam-electric installation. The foundation consists of 172 fir piles 40 ft. long, spaced 3 ft. on centers and cut off at El. 3, referred to the zero of low water established by the U. S. Coast and Geodetic Survey. The range of tide is from 7 to 8 ft. and the piles are never subjected to alternation of dryness and moisture, because the low velocity of water through the surrounding sand prevents a drop in the water plane below El. 5.



VIEW OF DERRICK AND RIG



JAW RIVETER FOR 100-FOOT STEEL STANDPIPE AT LONG BEACH

The concrete foundation, a 1:3:6 mixture, incloses 1 ft. of the top of the piles and ends at El. 13, the general level of the surface of the ground. The center of the foundation is cored out and filled with sand. The sand was water-tamped, weepers closed up later being left to take care of the excess water, and the whole area was covered with a 6-in. slab of concrete. After riveting, the bottom of the standpipe was lowered, and rests on a 1½-in. layer of 1 part cement and 3 parts sand mixed and laid dry on the 6-in. slab. It is expected that the mixture in the humid air of the coast will take up sufficient water to form a mortar.

Twelve 1¾-in. bolts 10 ft. long with 10 x 10 x ½-in. plate washers are set in the concrete and project 2 ft. above, fitting into lugs riveted to the sides of the standpipe. The bottom course of the shell, 6 ft. 2 in. high, is carried on a 4 x 4 x ⅝-in. angle riveted to the bottom, and the open top is finished with a 4 x 4 x ½-in. angle. Above the 6 ft. 2 in. bottom course are fifteen courses 6 ft. 2¾ in. high, made up of plate varying in thickness from ⅝ to ¼ in.

The erection is accomplished without scaffolding; one of the photographs shows the first ring with an Allen jaw riveter in place; the detail of the butt strap riveting and anchor lugs is well defined. The other picture shows progress in the erection, as well as the derrick and rig with which all material is handled. The inside of the standpipe is painted with Bitumastic solution and enamel; the outside with the solution only. The material is being furnished and erected by the Security Construction Company under the direction of W. A. Brackenridge, vice-president and general manager, and H. W. Dennis, construction engineer of the Southern California Edison Company.

GOVERNMENT railway construction to the extent of 208 mi. is under way in Chile, according to a recent consular report. The estimated cost of this work is $8,634,000. Of this 63 mi. are expected to be completed this year, at a cost of $1,848,000. The Government will then own and control 2270 mi.

Letters to the Editor

Design of Elevated Water Tanks

SIR: The figures given by Mr. Pillsbury on page 514 of the Engineering Record of May 2, for the relative stresses in elliptical-bottom and hemispherical-bottom tanks were so at variance with what the writer expected that he has obtained from the "Manufacturers' Catalog" the dimensions of the two 100,000-gal. tanks used for comparison.

Fig. 1 gives their principal dimensions. The elliptical bottom having a semi-major axis of $D/2$ and a semi-minor axis of $D/4$ has a radius of curvature at the lowest point of $(D/2)^2 \div (D/4) = D$. The pressure per square inch at this point is $23.17 \times 62.5/144 = 10$ lb. Applying this value in equation $T = PR/2$ we get $T = D \times 10/2 = 344 \times 10/2 = 1720$ lb. per linear inch, instead of 1300, as mentioned in Mr. Pillsbury's letter. Comparing this with the 1100 lb. per linear inch for the hemispherical bottom we find that it exceeds the latter by about 55 per cent.

If the 50,000-gal. tanks had been selected for comparison the difference would have been greater. Fig. 2 shows the dimensions of these tanks as given in the catalog. For the elliptical bottom $P = 19.5 \times 62.5/144 = 8.46$ lb. per square inch and $T = 264 \times 8.46/2 = 1117$ lb. per linear inch. For the hemispherical bottom $P = 27 \times 62.5/144 = 11.72$ lb. per square inch and $T = 228 \times 11.72/4 = 668$ lb. per linear inch. In this case the tensile stress in the elliptical bottom exceeds that in the hemispherical bottom by 67 per cent.

The writer believes it would be more consistent to compare two tanks of equal capacities and diameters as indicated in Fig. 3. For the elliptical bottom $P = 37 \times 62.5/144 = 16.1$ lb. per square inch and $T = 264 \times 16.1/2 = 2125$ lb. per linear inch. For the hemispherical bottom $P = 39 \times 62.5/144 = 16.93$ lb. per square inch and $T = 264 \times 16.93/4 = 1117$ lb. per linear inch. By comparing these two values it is seen that the tensile stress in the elliptical bottom exceeds that in the hemispherical bottom by 90 per cent.

Referring again to Fig. 1 and to equation (9) as given in Mr. Pillsbury's article on page 390 of the Engineering Record for April 4, the radius of curvature of the ellipse at the extremity of the major axis (i. e., at the connection of the bottom with the cylinder) is $(D/4)^2 \div (D/2) = D/8$. Substituting this value for r'' and $D/2$ for r' in equation (9) we get $T = P\,[r' - (r'^2/2r'')] = P[(D/2) - (D/2)^2/(2D/8] = P[(D/2) - D] = -PD/2$. Inserting the numerical values we get $P = 16 \times 62.5/144 = 6.94$ lb. per square inch and $T = -6.94 \times 344/2 = 1194$ lb. compression per linear inch.

GEO. L. CHRISTY.

White Plains, N. Y.

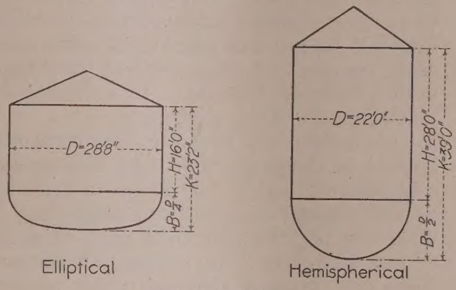
[Copy of the foregoing letter was submitted to Mr. Pillsbury, whose reply follows.—EDITOR.]

SIR: Mr. Christy makes the point that in general the stresses in the elliptical are considerably higher than in the hemispherical bottom. So far as the tank made by the one manufacturer mentioned is concerned, Mr. Christy's conclusions are modified by two facts:

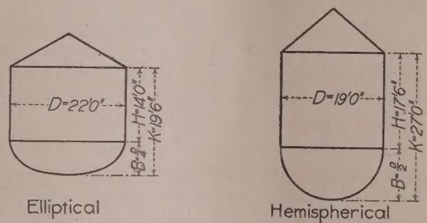
1. The radius connecting the bottom to the sides is made larger than $D/8$ and the radius at the lowest point is ordinarily made less than D. The resulting curve follows closely the outline of a semi-ellipse, but the calculated stresses are less.

2. A large steel riser pipe is riveted directly to the tank bottom, which is flat at the point of connection, and itself takes care of any expansion in the riser pipe. This steel supporting column in the center carries a part of the water load.

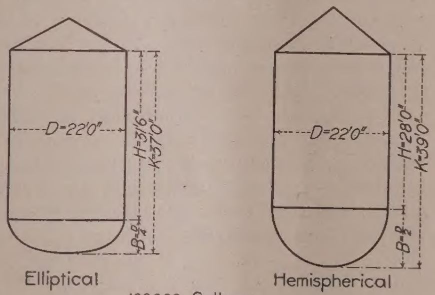
Even if as actually constructed the stresses were as stated by Mr. Christy, which they are not, the most important advantages of the construction would still exist. The elliptical bottom tank with the large riser was originally designed for railroad service. The form of bottom is adapted to a tank with large diameter and small total depth with the small consequent variation in pressure. Furthermore, the large riser pipe acts as a settling basin so as to facilitate cleaning, and it is made large enough in diameter to prevent, under working conditions, interruption of service from freezing, so that no wood frost boxing is required around the piping.



100000 Gallons
Fig.1



50000 Gallons
Fig. 2



100000 Gallons
Fig. 3

COMPARATIVE FIGURES OF WATER TANKS

The greatest advantage lies in the fact that this is the only type of tank which does not require an expansion joint. Other manufacturers recognize the advantages of the construction and imitate the design as closely as possible. In order to evade patents held by George T. Horton they are obliged to put in an expansion joint and, therefore, they make a virtue of necessity and claim that one is required in the elliptical bottom. Actually, the elliptical bottom being practically flat at the point of connection, takes care of the expansion in the riser pipe.

C. S. PILLSBURY,
Engineer, Chicago Bridge & Iron Works.
Chicago.

How the Ancients Would Have Controlled the Mississippi

SIR: In your issue of June 27, page 723, appears an "Abstract of a Paper by Sir William Willcocks, Read May 13 before the Engineers' Society of Western Pennsylvania." This paper deals solely with the flood problem of the lower Mississippi River, and its appropriateness in the above-mentioned connection is not quite obvious. Letting that pass, however, and going to the merits of the paper itself, it seems proper that some American engineer should make a reply to Sir William Willcocks, and while it is not an agreeable task to the writer, who feels conscious that it could be better done by some other engineer, yet no other has so far come forward for what must be regarded as an ungracious performance, probably from a reluctance to assume the rôle of critic of so distinguished a foreign visitor as Sir William.

With the above prelude the writer will offer some reflections of his own on Sir William's paper The first impression made by a perusal of the paper is of the serene assurance that characterizes his utterances.

Sir William Willcocks is generally recognized as an eminent and accomplished engineer in his own domain. He comes to the United States, into a field wholly different from that to which he has been accustomed, makes a cursory examination of what is much the greatest flood-control problem that the world presents as of present practical interest, calmly assumes an eminence of superiority, which treats the whole body of American engineers as kindergarten pupils, and proceeds to teach them their A B C's.

However skilful Sir William may have proved himself in handling the comparatively small problems of flood control, from the standpoint of an irrigation engineer, of the Nile, Euphrates, and Tigris Rivers, it is not apparent that he brings any message to American engineers in the solution of their much greater and wholly different problem of regulating the flood flow in the lower Mississippi River. And it seems mildly absurd that the engineering feats of the ancient Egyptians and Babylonians should be pointed to for the guidance of modern American engineers.

Sir William undertakes to tell us what the Babylonians would have done if they had been confronted with the Mississippi River problem, by way of rebuke to our stupidity. As these ancients never encountered any such conditions, either for magnitude or other analogies, it is not clear how their treatment of the case can be pronounced upon with such assurance. Perhaps they might have done some or all of the things that are suggested, if allowed some centuries, or a thousand years or so, in which to do them.

On the other hand, the partial reclamation of the lower Mississippi valley has, during the past half century, developed industries and values to the amount of many billions of dollars, which would otherwise have lain dormant in the womb of Time for some more or less indefinite future awakening. Moreover, the more complete reclamation of this territory is continually progressing, and under the stimulus of improved flood protection in recent years many thousands of acres have been added to the area under cultivation.

Sir William points to the large proportion of the area of the lower valley that still remains as "primitive forest," and asks what is the use of protecting it. There are two reasons why these lands are in a state of primitive forest—first, lack of assured protection from overflow, and, second, lack of adequate drainage.

The forested lands are the lowest lands and must catch any flood water that invades the country. Adequate drainage must follow assured protection from the river floods; and when that has been secured these lands will speedily come into the market and as speedily develop high values, with a corresponding increase in population and industrial activities. With increasing confidence in protection from the Mississippi floods, the cultivated area in the upper Yazoo basin has been fully doubled in the last twenty years, and the sum total of property values has increased about 400 per cent.

Sir William says that the difference between the Mississippi problem and those with which he is familiar is merely one of magnitude. While there are other important differences, that of magnitude alone may and does in many cases constitute, in the vernacular, "all the difference in the world." For example, compare the Quebec Bridge problem with that of bridges one-fourth its size.

The Mississippi River problem is peculiarly an American problem, and is unique in magnitude and other important features. American engineers have diligently studied this problem in all its phases for a longer period of time than would cover Sir William's entire professional career.

American engineers fully understand their problem and how it should be handled, and have no need to go to school to Babylonish engineers of prehistoric times to learn engineering science and practice; but they lack one advantage that appears to have been at the service of the ancients—to wit, the unlimited command of labor and money for their enterprises, as well as apparently unlimited time for their accomplishment. Given sufficient and timely provision of money, and the work of flood control, channel regulation and the development of navigation possibilities in the lower Mississippi River, like the Panama Canal, would in a comparatively short time emerge from the hands of the engineers as a completed product.

This problem is not one of complexity, nor of special difficulty, and may be presented in a few cardinal features, stated in the order of their relative importance, as follows:

1. Protection of caving banks from further erosion of the current.
2. Enlarging existing levees to grades and dimensions adequate to contain the greatest floods to be anticipated in the future.
3. Safeguarding the foundations of the levees against the flow of underground water under hydrostatic pressure and its eruption in the form of "geysers" behind the levee.

This last provision is readily accomplished by the building of small "sub-levees" behind the main levee, in places where the need is apparent, by which means water is ponded over treacherous areas, reducing the hydrostatic head and providing a "cushion" of dead water over the orifice of the "boils" or "geysers."

This treatment has proved in practice a complete and efficient remedy for the evil to be combated, the cost of which for the entire system of levees should be less than ½ per cent of the cost of the main levees.

Sir William Willcocks is eminently right in one of his pronouncements—to wit, in assigning fundamental importance to the protection of caving banks, a feature which is indeed the sine qua non of any plan for river regulation. A very large proportion of the millions of dollars that have been expended for the maintenance of levees has been engulfed by the river; and there can be no permanence of levee locations so long as caving banks continue unrestrained—although that is only one of the functions of bank protection, while there are others more fundamental in character. If American engineers are open to criticism concerning their treatment of this problem, it is for yielding to popular clamor for the immediate construction of levees and permitting without protest that work to dominate the primal need of bank protection.

This matter has, however, been controlled largely by Congress, that body making specific appropriations for levees in response to the popular demand and neglecting to make like provision for bank protection, for which there has been no popular demand. Federal engineers should, in the writer's opinion, have exerted their influence in the latter direction.

American engineers possess all necessary information for carrying out the project of flood control and channel regulation in the lower Mississippi River and lack only the needed funds for its accomplishment.

Provision was made in what is known as the Ransdell-Humphries bill for carrying forward this work in an orderly and effective manner, had the bill become a law. But doubts and confusion in the minds of Congressmen were implanted, and a great diversion was created by the glamour of that stupendous "rainbow chaser," the Newlands bill, which has hindered any legislation for this work; and the utterances of Sir William Willcocks, here commented on, which by inference assume that the engineers to whose hands this work is intrusted are all ignoramuses, must further militate against a return of the Congressional mind to a normal and sensible attitude toward this question.

T. G. DABNEY,
Chief Engineer, Yazoo-Mississippi Delta Levee District.
Clarksdale, Miss.

Girderless Concrete Slabs

SIR: I have read the article in the Engineering Record of June 27, page 731, by Mr. Krippner on girderless concrete slabs. The analysis has the merits of simplicity and of giving a conservative coefficient of bending moment, but I think it unfortunate to consider as applicable to design an analysis which assumes that a "slab separated from the adjacent slab and supported by a column at the center" can have a structural action which is the same as that of a slab which is continuous at the center with the adjacent slab. Inasmuch as Mr. Krippner later refers to reports of various boards of examiners which recommend coefficients of moment at the center of about $WL/25$, it appears that he must have in mind that there is a moment at the center, although under his assumption of a cantilever slab there could be none. I have looked for a typographical error somewhere the correction of which would set things right, but so far have found no place for one. The error seems to be fundamental and seems to follow the analysis through. Will the writer of the article furnish information which will straighten out the difficulty?

W. A. SLATER,
Department of Theoretical and Applied Mechanics, University of Illinois.
Urbana, Ill.

[This letter was submitted to Mr. Krippner with the request that he furnish the information asked for. His reply follows.—EDITOR.]

SIR: Replying to the letter of W. A. Slater, I wish to call your attention to the assumptions in my article. I first assume a panel disconnected from the floor construction and entirely free, as Mr. Slater mentions. The coefficients thus derived represent truly the total bending moment around the column capital for the entire panel. So much is true beyond dispute. My next assumption is based on the action of continuous beams fixed at support. It is shown in standard books on strength of materials that the bending moment at the support for such a beam is $WL/12$ and at the middle $WL/24$. This result may be obtained by considering the beam split in the middle and computing the bending moment on the cantilever part only. The load on this part is $W/2$ and the moment arm $L/4$. Hence the bending moment is $(W/2) \times (L/4) = WL/8$.

If we now consider the same beam joined in the middle, this total bending moment will divide itself into a positive moment at the center and a negative moment at the support. In other words, the moment at the support will be relieved to some extent by reason of the positive moment set up in the middle. This division of moments between the middle and the support is in the ratio of one to two, or the positive moment is one-half the negative, as may be seen above. As there is no good reason why the two together should not equal the total moment at the support when the beam is considered as a cantilever beam only, we have the following relation: Bending moment at middle of a continuous beam fixed at supports plus bending moment at support equals total bending moment. In other words, the moment at support equals

$$(WL/8) \times (2/3) = WL/12$$

and the moment at middle

$$(WL/8) \times (1/3) = WL/24$$

as referred to above.

Applying the same line of reasoning to the coefficients derived in my article, it follows that two-thirds of each represents the negative moment around the head of the column and one-third the positive moment at center. That the results should be lower than for a fixed beam is natural enough, considering the length of support in each case, which in one case is the full width of beam, or the whole panel length; in the other, only the circumference of the column capital, which is a function of the diameter.

JOHN KRIPPNER,
Architectural Engineer, Department of Buildings.
Chicago.

SEEPAGE WATER from the lower Otay dam at San Diego, Cal., is to be collected and pumped back into the reservoir. According to the annual report of Major Herbert R. Fay, superintendent of the water department, this will necessitate the building of a small crib or earth dam about ¾ mile below lower Otay reservoir for the purpose of impounding a day's supply of seepage water.